TA705.3G8L682002bc.1

# GEOTECHNICAL ENGINEERING REPORT

# LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT TASK 2.4.6 – BAYLANDS SANTA CLARA COUNTY, CALIFORNIA

*Prepared for*
CH2MHILL
1737 North First Street, Suite 300
San Jose, CA 95112

SANTA CLARA VALLEY WATER DISTRICT
LIBRARY
5750 ALMADEN EXPRESSWAY
SAN JOSE, CALIFORNIA 95118

November 8, 2002

**URS**

55 South Market Street, Suite 1650
San Jose, California 95113

TA
705.3
G8.L68
2002b
c.1
LIBRARY

Project 95-00112030.00 / 28648808

URS

November 8, 2002
Project 95-00112030.00 / 28648808

Mr. David Von Rueden
CH2MHILL
1737 North First Street, Suite 300
San Jose, CA 95112

Subject: Geotechnical Engineering Report
Lower Guadalupe River Flood Control Project
Task 2.4.6 Baylands
Santa Clara County, California

Dear Mr. Von Rueden:

In accordance with our Purchase Order/Subcontract 51347 dated August 14, 2001, we are pleased to submit this geotechnical engineering report for the Lower Guadalupe River Flood Control Project, to be constructed in Santa Clara County, California. This report addresses the portion of the project alignment between the Alviso Slough and the Union Pacific Railroad (UPRR), and is supplemental to our previous engineering report entitled "Geotechnical Investigation Lower Guadalupe River Flood Control Project," dated April 28, 2000.

The accompanying report presents our opinions and recommendations regarding the geotechnical aspects influencing design and construction of the proposed levee improvements. The opinions and recommendations have been based upon review of available information, the results of a field investigation, laboratory testing, engineering analyses we conducted as part of our scope, as well as engineering judgment and local experience.

Mr. Thomas Pennington, Staff Engineer, assisted in the selection of the engineering parameters, the engineering analysis of levee stability, potential levee seepage, levee settlement, and as well as preparation of the report. Mr. Michael L. Larson, Consulting Geotechnical Engineer, provided peer review for all geotechnical aspects of the project. This peer review was in accordance with the URS Corporation Quality Control Plan.

Since our report of August 5, 2002 was issued, we received review comments from CH2MHILL design team members on September 5, 2002 and met with the team members on September 18, 2002 to discuss the comments. We also received comments from the Santa Clara Valley Water District staff on October 21, 2002. On September 17, 2002, we received a set of plans stamped "Preliminary, Current as of September 17, 2002." Subsequently, we have incorporated all review comments and update the geotechnical design parameters consistent with the latest project data we have received.

URS Corporation
55 South Market Street, Suite 1650
San Jose, CA 95113
Tel: 408.297.9585
Fax: 408.297.6962

We are pleased to have been of service to you on this project. If any questions should arise, please contact our office.

Sincerely,

Stephen Huang

Stephen Huang, G.E. 02150
Deputy Project Manager

Paul J. Boddie

Paul J. Boddie, G.E. 00152
Project Manager

LAM/MLL:cal

REGISTERED PROFESSIONAL ENGINEER
S. STEPHEN HUANG
No. GE 002150
3-31-04
GEOTECHNICAL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
PAUL J. BODDIE
152
Exp. 3-31-06
GEOTECHNICAL
STATE OF CALIFORNIA

# Table of Contents

**SECTION 1 INTRODUCTION ........ 1-1**

1.1 Project Description ........ 1-1

1.2 Scope Of Work ........ 1-3

1.3 Previous Investigations ........ 1-4

1.4 Data Review ........ 1-5

**SECTION 2 SITE GEOLOGY AND SEISMICITY ........ 2-1**

2.1 Geologic Setting ........ 2-1

2.2 Regional Faults And Seismic Sources ........ 2-1
- 2.2.1 San Andreas Fault ........ 2-1
- 2.2.2 Hayward Fault ........ 2-2
- 2.2.3 Hayward Fault - Southeast Extension ........ 2-2
- 2.2.4 Calaveras Fault ........ 2-3
- 2.2.5 Monte Vista - Shannon Fault ........ 2-3
- 2.2.6 Sargent - Berrocal Fault ........ 2-4

2.3 Ground Motions ........ 2-4

2.4 Summary ........ 2-4

**SECTION 3 SITE AND SOIL CONDITIONS ........ 3-1**

3.1 Surface Conditions ........ 3-1
- 3.1.1 Field Reconnaissance ........ 3-1
- 3.1.2 General ........ 3-1
- 3.1.3 Segment Santa Clara County Marina Parking Lot to UPRR Tracks- Station 6+100 to Station 7+008 ........ 3-2
- 3.1.4 Segment at West Levee (Alviso) Overflow Weir ........ 3-2

3.2 Subsurface Conditions ........ 3-3
- 3.2.1 Field Exploration ........ 3-3
- 3.2.2 Laboratory Testing ........ 3-3
- 3.2.3 Soil and Groundwater Conditions ........ 3-4
  - 3.2.3.1 East Levee ........ 3-4
  - 3.2.3.2 West Levee Overflow Weir ........ 3-5
  - 3.2.3.3 River Channel ........ 3-5

**SECTION 4 LEVEE SLOPE STABILITY EVALUATION ........ 4-1**

4.1 Cross Sections Selected For Analysis ........ 4-1
- 4.1.1 Selection of 23 Levee Cross Sections ........ 4-1
- 4.1.2 Selection of Two Levee Cross Sections for Task 2.4.6 ........ 4-1
- 4.1.3 Selection of Shear Strength Parameters ........ 4-2
  - 4.1.3.1 Cohesive Soils in Levee and Levee Foundation ........ 4-2
  - 4.1.3.2 Cohesive Soil in Channel ........ 4-4
  - 4.1.3.3 Granular Soils ........ 4-5

4.2 Method of Analysis ....4-5
4.2.1 Pore Pressures ....4-6
4.2.2 Static Loading Conditions ....4-6
4.2.2.1 End of Construction ....4-6
4.2.2.2 Sudden Drawdown ....4-7
4.2.2.3 Steady Seepage From Full Flood Stage ....4-7
4.2.3 Earthquake Loading Conditions ....4-7
4.3 Stability Results of Levee Sections ....4-8

**SECTION 5 SETTLEMENT EVALUATION ....5-1**

5.1 Subsidence ....5-1
5.2 Levee Settlement ....5-1
5.2.1 Inferred Settlement ....5-1
5.2.2 Predicted Settlement ....5-1
5.3 Future Settlement of East Levee ....5-2
5.4 West Levee Overflow Weir ....5-3

**SECTION 6 SEEPAGE EVALUATION ....6-1**

6.1 Method Of Analysis ....6-1
6.2 Cross Sections And Results Of Analysis ....6-1

**SECTION 7 FLOODWALL DESIGN PARAMETERS ....7-1**

7.1 Background ....7-1
7.2 Corps Requirements ....7-1
7.3 Wall Settlement ....7-2
7.4 Driving Side Design Earth Pressure ....7-3
7.5 Resisting Side Design Earth Pressure ....7-3
7.5.1 At Rest Earth Pressure ....7-3
7.5.2 Passive Pressure ....7-4
7.6 Bearing Capacity ....7-4
7.7 Foundation Uplift ....7-4
7.8 Drilled Piers ....7-5
7.9 Lateral Load Capacity of Drilled Piers ....7-5
7.10 Construction Considerations for Drilled Piers ....7-6
7.11 Corrosion Investigation ....7-7

SECTION 8 LIQUEFACTION POTENTIAL ........ 8-1

SECTION 9 SITE PREPARATION AND EARTHWORK ........ 9-1

9.1 General ........ 9-1
9.2 Clearing and Demolition ........ 9-1
9.3 Grubbing ........ 9-1
9.4 Stripping ........ 9-1
9.5 Removal of Sediment in Channel ........ 9-1
9.6 Excavation ........ 9-2
9.7 Subgrade Preparation ........ 9-2
9.8 Fill Materials ........ 9-3
  9.8.1 Approval ........ 9-3
  9.8.2 Onsite Materials ........ 9-3
    9.8.2.1 Channel Sediment ........ 9-3
    9.8.2.2 Excavation for Floodwalls ........ 9-4
    9.8.2.3 Existing Armoring Aggregate ........ 9-4
    9.8.2.4 Excavation for West Levee Overflow Weir ........ 9-4
  9.8.3 Import Materials ........ 9-4
    9.8.3.1 Levee and Weir Fill ........ 9-4
    9.8.3.2 Bridging Material ........ 9-5
    9.8.3.3 Armoring Aggregate ........ 9-5
9.9 Fill Placement and Compaction ........ 9-5
9.10 Construction Dewatering ........ 9-6

SECTION 10 CONCLUSIONS AND RECOMMENDATIONS ........ 10-1

10.1 General ........ 10-1
10.2 Results of Levee Slope Stability Evaluation ........ 10-1
10.3 Results of Settlement Analysis ........ 10-1
10.4 Results of Seepage Analysis ........ 10-2
10.5 Site Preparation and Grading ........ 10-2
10.6 Liquefaction Potential ........ 10-2

SECTION 11 LIMITATIONS ........ 11-1

SECTION 12 REFERENCES ........ 12-1

# SECTION 1

This report presents the results of the geotechnical investigation for the Baylands portion of the Lower Guadalupe River Flood Control project for Levee Improvements, including the portion of the Alviso Slough north of the Union Pacific Railroad (UPRR). The location and layout of the project are shown in Figure 1, Site Map. A more detailed layout of the entire project, including the approximate centerline of the existing West Levee and East Levee, is shown in Figure 2, Site and Field Exploration Location Plan. This figure includes the design control line with stationing presented in meters, as well as estimated limits of historical meanders and locations of existing and new field exploration points. Locations of streets and highways crossing the Guadalupe River are also shown in Figure 2. Topographic maps with aerial photography background were presented as part of our April 28, 2000 report for this project. Throughout the text of this report, elevation, distance and stationing will be presented in metric units; except for stationing, English units will also be presented in parenthesis. According to the latest design drawings with sheets dated June 28, July 1 and July 3, 2002 (referred to as July 2002 Design Drawings from this point onwards), there are three separate stationing systems, including the “P” Line, “E” Line and “W” Line. The “P” Line traverses the approximate centerline of the existing river channel; the station numbers presented in this report are based on the “P” Line unless otherwise stated. The “E” Line traverses the crest of the existing East Levee, i.e. looking upstream. The “W” Line traverses the crest of the West Levee, i.e. looking upstream. However, all graphs, charts and laboratory test results are presented in English units.

The report presents the results of (1) a review of the available subsurface information, construction records and as-built drawings, (2) the subsurface exploration and laboratory testing program conducted for this study, and (3) the results of engineering evaluations that serve as the basis for our opinions and recommendations regarding the geotechnical engineering aspects of the project.

The goal of this study is to provide a design level geotechnical investigation for the Lower Guadalupe River in order to bring the levees into compliance with the requirements of the Federal Emergency Management Agency (FEMA) and the United States Army Corps of Engineers (Corps). Ultimately, the Santa Clara Valley Water District (District) wishes to obtain FEMA certification of the levees in order to receive a Letter of Map Revision (LOMR).

To meet FEMA standards, the levees must provide a minimum freeboard of 0.9 meter (3 feet) above the 100-year (1-percent) flood. An additional 0.3 meter (1 foot) above the minimum is required within 30.5 meters (100 feet) on either side of structures (such as bridges) riverward of the levees or wherever the flow is constricted. An additional 152 mm (1/2 foot) above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required. The District also must demonstrate that the levees are stable from a geotechnical standpoint.

## 1.1 PROJECT DESCRIPTION

The Lower Guadalupe River has been defined as that portion of the river that extends from I-880 to the river’s terminus with the San Francisco Bay at Alviso. This portion of the Guadalupe River is located in the City of San Jose and Santa Clara County, California. This report addresses the portion of the levee alignment between the Alviso Marina (Station P6+130) to the UPRR (about Station P7+008), for a total length of about 878 meters (2,881 feet). According to Sheets C-1A, C-2A, and C-3A, the existing East Levee will be raised a maximum height of 1.2 meters (4.0 feet) with landside and riverside slopes of 2:1 (horizontal to vertical). Crest width varies between 3.5 and 5.5 meters (11.5 and 18 feet).

According to sheets S-3 and S-23 of the July 2002 Design Drawings, floodwalls are planned with the lengths and heights as described in the following table.

| Floodwall | Station | Location | Length (m) | Wall Height (mm) | | Sheet Number | Maximum Fill Height (m) |
|---|---|---|---|---|---|---|---|
| | | | | Minimum | Maximum | | |
| FW6E | E20+407.469 to E20+461.907 | East Levee | 54.467 | 1,000 | 1,000 | S-23 | 0.7 |
| FW1E | E20+975.300 to E20+986.648 | East Levee | 19.754 | 1,500 | 2,100 | S-3 | 0 |

Based on the above table, the total length of the floodwalls is 74.2 meters (244 feet); H is defined from top of floodwall to top of footing. Since the minimum burial depth is 0.75 meter (2.5 feet), the wall height above finished grade ranges from 0.25 to 1.35 meters (0.8 to 4.4 feet). There is about 0.7 meter (2.5 feet) of fill planned towards the ends of floodwall FW6E to transition from raised levee to floodwalls.

No retaining walls or Mechanically Stabilized Embankment (MSE) walls are planned within this portion of the alignment.

A maintenance road and turnaround is planned on the East Levee between Stations E20+867 and E20+975; it will be 5.5 meters (18 feet) in width.

According to Sheets CD-1 through CD-3, a weir structure (West Levee Overflow Wier) is planned west of the Guadalupe River Channel between about Station P6+462 and P 6+818. According to Figures 3, 4, and 5 (corresponding to sheets CD-1, CD-2, and CD-3 revised from CH2MHILL dated June 3, 2002) the weir is approximately 356 meters (1,168 feet) long. It will be raised over an existing weir and will be trapezoidal shaped in cross section. Height of the weir will vary, with finished grade ranging from Elevation 3.13 to 4.20 meters (10.3 to 13.2 feet). Maximum fill height is approximately 2.3 meters (7.5 feet). Side slopes will be 4 to 1 (horizontal to vertical) on the inboard and outboard faces. As shown in Figure 5, the raised weir will be covered with an articulated concrete mat and the crest will also be surfaced with 0.3 meter (12 inches) of aggregate base.

As in the previous study (URS, April 2000), we have used the NGVD survey datum for the elevation of the ground surface at all explorations and subsurface profiles; both

NGVD English and Metric units are provided in this report text. In the event that conversion to NAVD is desired in metric units, the following equation should be used:

NAVD (metric) = NGVD (metric) + 0.81

The elevations shown in the August and December 2001 drawings are based on the NAVD datum.

The 100-year (1-percent) flood hydrograph provided by the District and developed by the Corps is shown in Figure 6. As shown, the peak discharge is 481.4 cubic meters per second or cms (17,000 cubic feet per second or cfs); the duration of flow exceeding 453 cms (16,000 cfs) only lasts about 12 hours. When the discharge rate is reduced to 113 cms (4,000 cfs) or about one-quarter of the peak discharge, the time interval is about 42 hours.

## 1.2 SCOPE OF WORK

The following scope of work has been performed to serve as a basis for the development of the opinions and recommendations presented in this report:

- Reviewed available information including the following:
  - Current levee cross sections, topographic drawings, and design drawings provided by the District and CH2MHILL.
  - Proposed improvements including raised East Levee and West Levee Overflow Weir.
- Selected locations of new borings.
- Cleared utilities at field exploration points through Underground Service Alert.
- Performed field exploration including:
  - 7 borings to depths of 10.2 to 24.8 m (33.5 to 81.5 feet); boreholes were grouted upon completion.
  - Includes Borings EB-27, EB-28, CB-14, CB-15, CB-16, CB-17, and CB-18.
- Performed laboratory testing, including:
  - Grain size distribution
  - Plasticity index
  - Unconfined compression, water content and dry density
  - Consolidation
- Identified key geotechnical cross-sections along East Levee.
- Characterized soil profiles for these cross-sections.
- Identified potential geotechnical issues.
- Selected two (2) cross-sections judged to be most critical.
- Performed geotechnical calculations:

  - At two levee cross sections, both on the East Levee
  - Factors of safety for slope stability for 4 loading conditions
  - Slope stability earthquake deformations
  - Settlement at both cross sections on East Levee and West Levee Overflow Weir
  - Seepage
  - In accordance with Corps Engineer Manual No. EM 1110-2-1913 (Design and Construction of Levees) and Corps Engineer Manual No. EM 1110-2-1904 (Soil Mechanics Design - Settlement Analysis)
  - Floodwall design parameters
- Prepared geotechnical report:
  - Summarized the results of all the above tasks
  - Developed geotechnical design parameters for the reinforced concrete floodwalls
  - Developed geotechnical opinions and recommendations regarding the structural integrity of the East Levee.
  - Developed recommendations for site preparation and earthwork

## 1.3 PREVIOUS INVESTIGATIONS

We have previously performed an investigation for the subject project. The results of that investigation were presented in a report entitled "Geotechnical Investigation, Lower Guadalupe River Flood Control Project, Santa Clara County, California," dated April 28, 2000. That report included the entire alignment of the project from the Union Pacific Railroad (UPRR) to I-880. We performed slope stability, seepage, and settlement analyses, and provided recommendations based on the selection of seven critical cross sections.

The logs of 185 previous borings completed along the levees or for projects in the immediate vicinity of the levees are provided in Appendix A of our previous report entitled "Geotechnical Investigation, Lower Guadalupe River Flood Control Project, Santa Clara County, California", dated April 28, 2000. The logs represent a total of 26 different projects, over the period of 1960 through 1997. The index list for each project is shown in Table 1; a different boring number index is assigned for each project. The locations of the previous borings and CPTs (along with the new borings) are shown in Figure 2, Site and Field Exploration Location Plan, and Figure 7, Boring Location Plan – West Levee Overflow Weir. The depth of these borings ranges from 2.4 to 30.8 meters (8 to 101 feet). Most of the existing boring logs are from investigations previously performed by URS' predecessor firm, Woodward-Clyde Consultants. The existing borings shown on Figure 2 were selected based on these two criteria:

- They were located within or in close proximity to the footprint of the levees; and
- The technical information provided on the log was reliable, in our engineering judgment.

It should be noted that borings in the initial investigations in the early 1960s were drilled prior to construction of the levees and therefore reveal subsurface conditions only in the foundation. Subsequent borings drilled mostly in the 1980s and 1990s extended through the levee section and therefore revealed the levee fill and underlying foundation conditions at that time.

Most of the routine laboratory test data are shown on the individual boring logs, i.e. water content, dry density, and unconfined compression. The results of more complex laboratory tests such as grain size distribution, plasticity index, consolidation and consolidated undrained triaxial compression (with pore pressure measurements) from the previous investigations are presented in Appendix B of the 2000 report.

## 1.4 DATA REVIEW

As part of the April 2000 investigation, we reviewed the available levee design and construction information, as well as related records, mostly provided by the District including the following:

- Drawings for the original 1963 river relocation
  - 16 sheets (some as-built) dated 1959 and 1960 and entitled "Map and General Construction Plans of Zone C-1 (Central) Project No. 3, Guadalupe River Improvement, San Francisco Bay to Branham Lane, Santa Clara County Flood Control and Water Conservation District, Santa Clara County California"; most drawings prepared by Creegan and D'Angelo.
  - 54 sheets (some as-built) dated April 26, 1963 and entitled "Map and General Construction Plans of Guadalupe River Unit 1 From Montague Road to Alviso Slough, Zone C-1 (Central), Project No. 30016, Santa Clara County Flood Control and Water Conservation District, Santa Clara County, California"; most drawings prepared by Creegan and D'Angelo.
- As-built drawings for the 1983 levee raising including 116 sheets dated September 27, 1984(as-built) and entitled "Map and Construction Plan for Guadalupe River From S.P.R.R. to Highway 101, Central Flood Control Zone, Project No. 3015, Santa Clara Valley Water District"; logs of 10 soil borings were included on 3 plan sheets.
- As-built drawings for the 1995 interim raising between UPRR and Highway 101, including 15 sheets dated April 11, 1996 (as-built) entitled "Map and General Plan for Guadalupe River From Southern Pacific Railroad to Highway 101, Central Flood Control Zone, Project No. 3015, Santa Clara Valley Water District".
- Approximately 240 current levee cross sections.
- Current topographic and aerial photography drawings including 15 sheets dated November 25, 1997; no title; from U.P.R.R. bridge to south of Highway 101 (Airport Island Bridge). Contours are plotted on both levees using a 1.0 meter contour interval.
- Existing elevations of crest of both East and West Levees. Settlement was estimated by subtracting as-built levee crest elevations from the existing elevations.

- A graphical plot of land subsidence from a United States Geological Survey paper entitled "Land Subsidence in the Santa Clara Valley, Alameda, San Mateo and Santa Clara Counties, California", 1971, by J.F. Poland.

We have also reviewed available District drawings and other historical records for locations of meanders. As shown on Figure 2 Site and Field Exploration Location Plan, the approximate channel limits of old meanders are plotted for almost the entire length of the project; these are based on District drawings dated 1960 and 1963. Between Tasman Drive and Montague Expressway, the locations of earlier meanders are based on a 1873 map obtained from the archives of the San Jose Historical Museum ("Map of a Re=Survey of the "Lick" Property, Party, Part of "Rancho Rincon De Las Esteros", surveyed October 31, 1873 by J.H. Pieper and Re=Surveyed July 1887 by H.A. Braissard) were identified and plotted on Figure 2.

During construction of the levees, we understand the District typically required the removal of soft deposits within the meander and within the footprint of the levee and replacement with soil spread in layers and compacted by track walking with a bulldozer or other track-mounted equipment. However, no specific relative compaction requirements were set.

As part of the current investigation, we reviewed the available levee improvement design information, mostly provided by CH2MHILL, including the following:

- "Santa Clara Valley Water District, Lower Guadalupe River Planning Study, Engineer's Report, Project No. 3015" by CH2MHILL, Inc. dated August 2001.
- "Santa Clara Valley Transportation Authority, Tasman Corridor Mitigation and Monitoring Plan, USACE File # 18881S92, Project No. 725-03" by H.T Harvey & Associates dated January 31, 1997. Appendix G of their report contains a report entitled "Geotechnical Investigation Report, Proposed Levee and Improvements, Guadalupe River Site, San Jose, California", by Geo/Resource Consultants, Inc. dated January 10, 1997 (Project Number 1935-000). Their field exploration program included two test pits (Test Pits A and B to maximum depth of 8.0 feet) and six borings (Borings B-1 through B-6) to depths of 25 to 35 feet. A copy of the field exploration plan, logs of borings, test pits and laboratory test results are included in Appendix A of this report. For their geotechnical analyses, the proposed levee had side slopes of 2 horizontal to 1 vertical, crest at Elevation 6.3 m (18.0 feet plus 2.66 feet which equals 20.66 feet) and crest width of 4.9 m (16 feet). They performed slope stability analyses for four loading conditions. In addition, they performed a settlement analysis of the levee. They estimated long term total settlement "ranging from 9 inches to a maximum of 3.6 feet. The majority of the settlement will occur within 25 years after construction.....In most of the levee alignment, however, we anticipate that the total settlement to be on the order of 1 foot."
- Preliminary drawings for the Mitigation Levee entitled "Tasman Corridor Project, Guadalupe Mitigation Site" dated January 16, 1997 by ANWest, Inc. for the Santa Clara Valley Transportation Authority; included are nine sheets: G-1 and civil sheets C-1 through C-8. These are included in Appendix A. These drawings indicate this

levee has side slopes of 2:1 (horizontal to vertical), a crest width of 5.5 m (18 feet) and a crest finished grade at generally Elevation 6.3 m (18.0 feet).

- "Supplemental Design Memorandum, 30-Percent Design Submittal, Lower Guadalupe River Flood Protection Project, Alviso Marina to Interstate 880, Project No. 3301511" by CH2MHILL, dated December 7, 2001.
- Drawings for the proposed levee improvements in the plans entitled "Map and Construction Plan for Lower Guadalupe River Flood Protection Project from the Alviso Marina to Interstate 880", Project No. 301511, dated December 7, 2001.
- Drawings for the proposed levee improvements in the plans entitled "Map and Construction Plan for Lower Guadalupe River Flood Protection Project from the Alviso Marina to Interstate 880", Project No. 301511, dated April 29, 2002.
- Final submittal drawings for the Baylands dated June 28 and July 1, 2002, including Sheet Numbers C-1 through C-4, C-1A through C-4A, Y-1, CD-1 through CD-4, S-3, S-23 and S-25 through S-28.

SECTION 2

This section was included in the April 2000 report, and is repeated herein for ease of reference. No additional geologic study was performed as part of the current investigation, except Section 2.3 "Ground Motion" has been modified.

## 2.1 GEOLOGIC SETTING

The levees span from the northerly end of the Santa Clara Valley in Alviso to nearly the heart of the Valley in San Jose; the project area lies between the Santa Cruz Mountains to the west and the Diablo Range to the east. The Santa Clara Valley is a broad northwesterly trending alluvial filled basin. The older Mesozoic rocks of the Santa Cruz Mountains and the Diablo range underlie the thick accumulations of alluvial sediments in the Santa Clara Valley. These alluvial sediments are divided into the lower older Santa Clara Formation, and the upper younger surficial deposits of alluvium, and alluvial fans.

Depth to bedrock is estimated to be at least 92 meters or 300 feet (Rogers and Williams, 1974) along the levee alignment.

## 2.2 REGIONAL FAULTS AND SEISMIC SOURCES

The active faults that could potentially generate earthquakes producing strong ground motions along the Guadalupe River Flood Control Improvement Project are described below and are shown on Figure 8, Regional Active Faults. Each fault is characterized in terms of the type of faulting, rupture length, dip, maximum magnitude, and slip rate. The 84th percentile peak horizontal accelerations potentially generated by the maximum earthquake for each seismic source are tabulated in Tables 2a to 2d. The maximum magnitudes for these faults were adopted from the Working Group on Northern California Earthquake Potential (WGNCEP, 1996).

### 2.2.1 San Andreas Fault

The San Andreas fault extends from the Gulf of California, Mexico, to Point Delgada on the Mendocino Coast in northern California, a total distance of 1,200 km (746 miles). The San Andreas accommodates the majority of the motion between the Pacific and North American plates. This is the largest active fault in California and is responsible for the largest known earthquake in northern California, the 1906 moment magnitude ($M_w$) 8 San Francisco earthquake (Wallace, 1990). Movement on the San Andreas fault is right-lateral strike-slip, with a total offset of some 560 km or 348 miles (Irwin, 1990). In northern California, the San Andreas fault is clearly delineated, striking northwest, approximately parallel to the vector of plate motion between the Pacific and North American plates. Over most of its length, the San Andreas is a relatively simple, linear fault trace. However, immediately south of San Francisco Bay, the fault splits into a number of fault splays, including the Calaveras and Hayward faults. In the San Francisco Bay area the main trace of the San Andreas fault forms a linear depression along the peninsula occupied by the Crystal Springs reservoirs. Geomorphic evidence for Holocene faulting includes fault scarps in Holocene deposits, right-laterally offset streams, shutter ridges and closed linear depressions (Wallace, 1990). The 1906 earthquake resulted from rupture of the fault from San Juan Bautista north to Point Delgada, a distance of approximately 470 km (293 miles). The average amount of slip on the fault during this earthquake was 5.1 meters (16.7 feet) in the area to the north of the Golden Gate, and 2.5 meters (8.2 feet) in the Santa Cruz Mountains (WGNCEP, 1996).

Based on differences in geomorphic expression, fault geometry, paleoseismic chronology, slip rate, seismicity, and historic fault ruptures, the San Andreas fault is divided into a number of fault segments. In the San Francisco Bay Area, these segments include the Santa Cruz Mountains, possible source of the 1989 $M_w$ 7.1 Loma Prieta earthquake and the Peninsula segment. The slip rate on these segments is 14 and 17 mm/year (0.55 and 0.67 inch/year) respectively. Both of these segments, along with the North Coast segment, ruptured during the 1906 $M_w$ 8 San Francisco earthquake. The Santa Cruz Mountains, Peninsula, and 1906 rupture segments have calculated maximum credible earthquakes (MCEs) of $M_w$ 7, 7, and 8, respectively (WGNCEP, 1996).

### 2.2.2 Hayward Fault

The Hayward fault extends for 100 km (62 miles) from the area of Mount Misery, east of San Jose, to Point Pinole on San Pablo Bay. Systematic right-lateral geomorphic offsets and creep offset of cultural features have been well documented along the entire length of the fault (e.g., Lienkaemper et al., 1991). The Hayward fault is considered to be the most likely source of the next major earthquake in the San Francisco Bay area (WGCEP, 1990). The last major earthquake on the Hayward fault, in October 1868, occurred along the southern segment of the fault. This Richter local magnitude ($M_L$) 7 event caused toppling of buildings in Hayward and other localities within about 5 km (3.1 miles) of the fault. The surface rupture associated with this earthquake is thought to have extended for approximately 30 km (18.6 miles), from Warm Springs to San Leandro, with a maximum reported displacement of 1 meter (3.28 feet). As well as moving during earthquake ruptures, the Hayward fault also moves by aseismic creep. Measurements along the fault over the last two decades show that the creep rate is 5 to 9 mm/year (0.20 to 0.35 inch/year) (Lienkaemper et al., 1991). The Working Group on Northern California Earthquake Potential (1996) assigns MCEs of $M_w$ 6.9 and recurrence intervals of 210 years for both the northern and southern segments of the Hayward fault. Recent research of historical documents has led to the conclusion that an earthquake in 1836, previously thought to have occurred on the northern Hayward fault occurred elsewhere, thereby increasing the time since the last earthquake on this segment of the fault. Recent paleoseismic trenching along the northern Hayward fault indicates that the last surface rupturing earthquake along this part of the fault was sometime between 1626 and 1724 (Lienkaemper et al., 1997). This study also indicated that there are at least four surface rupturing earthquakes in the last 2,250 years.

### 2.2.3 Hayward Fault - Southeast Extension

The Hayward fault zone extends to the south of Milpitas, along the eastern margin of the Santa Clara Valley, as a broad zone of reverse and reverse-oblique faulting. This zone of faulting includes the Quimby, Crosley, and Evergreen faults, as well as the southeastern extension of the Hayward fault. The predominant sense of motion on these faults is right-lateral strike-slip on the continuation of the Hayward fault itself, becoming reverse-oblique, with an increasing thrust faulting component towards the west. The thrust faults dip to the northeast. Trenching investigations and the geomorphic expression of these faults indicate that they have been active during late Pleistocene, and possibly even Holocene time. The Working Group on Northern California Earthquake Potential (1996) assigns a maximum earthquake of $M_w$ 6.5 to the southeastern extension of the Hayward fault.

### 2.2.4 Calaveras Fault

The Calaveras fault is a main component of the San Andreas system, branching off the main San Andreas fault south of Hollister, and extending northwards for approximately 120 km (75 miles) to die out in the area of Danville. The predominant sense of motion on the Calaveras fault is right lateral strike-slip. A smaller component of vertical displacement is evident in some areas along the fault trace. The Calaveras can be divided into two distinct sections, northern and southern, with the boundary located at Calaveras Reservoir.

The Calaveras fault has generated a number of moderate-size earthquakes in historic time, including: (1) the 1861 $M_L$ 5.9 event; (2) the 1886 $M_L$ 5.4 event; (3) the 1897 $M_L$ 6.2 event; (4) a probable $M_L$ 6.5 event in 1911; (5) the 1988 $M_L$ 5.1 Alum Rock event; (6) the 1979 $M_L$ 5.9 Coyote Lake event; and the 1984 $M_L$ 6.2 Morgan Hill event.

To the south of Calaveras reservoir, microseismicity clearly delineates the active trace of the fault. The long term slip rate and contemporary creep rate for the southern Calaveras fault are approximately 15 mm/year (0.59 inch/year) (WGNCEP, 1996). The Working Group for Northern California Earthquake Potential (1996) calculated a MCE of $M_w$ 6½ for the (southern and northern) Calaveras fault.

### 2.2.5 Monte Vista - Shannon Fault

The Monte Vista-Shannon fault zone is one of a series of southwest-dipping thrust and reverse-oblique faults along the southwestern margin of the Santa Clara Valley. Collectively known as the Foothills Thrust System, this is a series of range-front faults that mark the boundary between the southern Santa Cruz Mountains and the western Santa Clara Valley. The Monte Vista fault extends from Palo Alto, southeast, through Los Altos, Los Altos Hills and Cupertino to Saratoga. Rather than a single fault, this is a belt of southwest-dipping thrust faults. Limited trench data, stream channel profiles, and stream terrace elevations all indicate late Pleistocene and possibly Holocene movement on the Monte Vista thrust (Hitchcock *et al.*, 1994). To the south of Saratoga, the continuation of the Monte Vista fault is mapped as the Shannon fault. The Shannon fault also comprises a series of southwest-dipping thrust faults that offset Miocene bedrock and cut a paleosol reported to be less than 20,000-years-old [R. McLaughlin, U.S. Geological Survey, reported in Hitchcock *et al.*, (1994)]. The Monte Vista-Shannon fault is capable of generating a MCE of $M_w$ 6¾ (WGNCEP, 1996).

### 2.2.6 Sargent - Berrocal Fault

The Sargent-Berrocal fault is a 94-km-long (58.4 miles), complexly branching system of southwest-dipping thrust and reverse oblique faults associated with the San Andreas fault. Evidence of recent movement along the Sargent fault include reports of surface rupture along the fault trace in the vicinity of Page Mill Road in Palo Alto during the 1906 earthquake and some possible aftershocks of the 1989 Loma Prieta earthquake. The northern two-thirds of the fault converges with the San Andreas fault at depth, and may not be an independent seismic source (WGNCEP, 1996). The Sargent-Berrocal fault is considered capable of generating a MCE of $M_w$ 6¾.

## 2.3 GROUND MOTIONS

To characterize the attenuation of ground motions from the seismic sources (described above) to the Guadalupe River Flood Control Improvement Program, we have used empirical attenuation relationships appropriate for deep soil. These include Abrahamson and Silva (1997), Boore et al. (1993), Campbell (1997), and Sadigh et al. (1997). 84th percentile peak horizontal ground accelerations (PGAs) were calculated for each of the designated four reaches of the Guadalupe River Flood Control Improvement Project (Tables 2a to 2d). The limits of the four segments are summarized in the following table.

| Segment | From | To |
|---|---|---|
| 1 | Alviso Marina | Tasman Drive |
| 2 | Tasman Drive | Montague Expressway |
| 3 | Montague Expressway | Highway 101 |
| 4 | Highway 101 | I-880 |

The PGAs were calculated using the MCE for each seismic source described above, and the closest approach of each of these sources to each reach of the Project. For each site, the controlling seismic source is a $M_w$ 6.5 earthquake along the southeast extension of the Hayward fault. This event is capable of generating an 84th percentile peak horizontal ground acceleration of 0.55 g in Segment 1, which includes the levee limits of Task 2.4.6.

## 2.4 SUMMARY

There are no known active faults that cross the project alignment. However, the levees will be subject to strong ground shaking from known nearby active faults.

## 3.1 SURFACE CONDITIONS

### 3.1.1 Field Reconnaissance

The surface conditions will be described for the East Levee and the West Levee Overflow Weir, starting at the north end of the UPRR Bridge (Station 7+008) and proceeding north to the Alviso Marina (Station 6+130). The description of the surface conditions is based on our previous field reconnaissance during which we walked the entire alignment of both levees on December 19, 22, and 23, 1997 from Station 6+100 to Station 17+600. At that time, we observed surface conditions of the crest, both side slopes and the inboard and outboard toe of this levee. Our observations were documented in detail and are presented in Appendix D of the April 2000 report. This documentation also included areas of apparent distress such as surface cracking, wet areas, settlement, deformation and undermining of the levee toe. At the time of our field visit, the water flowing in the Guadalupe River was confined to the low flow channel; therefore, the water surface generally was below the level of the inboard toe. Furthermore, in June 2002 we performed a site reconnaissance of the West Levee Overflow Weir; our observations are described later in this section.

### 3.1.2 General

In general, the existing East Levee is of earthfill construction, is trapezoidal in cross section and ranges in height from about 2 ½ to 4 ½ meters (8 to 15 feet). The crest serves as an access road and is covered with armoring aggregate (gravel). In 1995 both levees between the UPRR and Highway 101 were raised, including some areas with soil reinforcement or concrete floodwalls, in many cases the top slopes were steepened. The entire 1995 crest was surfaced with armoring aggregate; the crest width is about 4 to 5¾ meters (13 to 14 feet). Numerous culverts and pipes also cross or penetrate the levee. Typically both the inboard and outboard slopes are vegetated with grass, weeds and scattered shrubs. Animal burrows are common along the entire alignment but are randomly located on the face of both side slopes. Access for vehicles is provided by the roadway on the crest and supplemented with ramps cut into the inboard slopes of both levees; at some locations an access road is also positioned just beyond the toe of the inboard slope. The low flow channel in some areas approximates the design centerline and in some areas meanders in close proximity to the inboard toe of the levee. Between the inboard toes of the levees, (i.e. in the channel area), the vegetation typically consists of weeds, grass and occasional trees.

Surface conditions for each segment are described in more detail in the following sections; refer to Appendix D of the April 2000 report for a more complete list of observations, including culverts and pipes observed during a site reconnaissance. For reference purposes for each segment, please refer to the topographic drawings in Figures 3a to 3o of the April 2000 report that include aerial photography.

### 3.1.3 Segment from Santa Clara County Marina Parking Lot to UPRR Tracks-Station 6+100 to Station 7+008

Within this segment the project is limited to the East Levee located near the town of Alviso. The East Levee begins at the Santa Clara County Marina parking lot and extends southerly through Alviso to the UPRR bridge. An abandoned building (Old Cannery) that extends into the outboard slope of the East Levee at two locations is positioned at about Station 6+270; this includes a brick retaining wall and a concrete retaining wall near the toe of the levee. Ponded water was observed (1) inside a portion of this structure on top of a concrete floor slab and (2) outside and east of this structure in a low area. Near Station 6+360 a reinforced earth wall retains the outboard slope of the levee; it appears to be in sound structural condition. An existing 2-story structure is located east of this wall. Near Station 6+180 a stockpile of dry mud was observed on the outboard slope. Within and adjacent to the river channel, numerous old boats are stored from about Station 6+200 to Station 6+800. Near Station 6+460, the Alviso Yacht Club Building is positioned on a berm extending east of the outboard slope. There are no structures abutting the outboard slope in this segment south of Station 6+460. However, the Town of Alviso lies immediately east of the levee; numerous houses and small businesses are prevalent in the area.

Scattered animal burrows were observed on both slopes throughout this segment; two clusters were noted.

### 3.1.4 Segment at West Levee Overflow Weir

Within this segment the project is limited to the West Levee Overflow Weir located in Alviso. The existing overflow weir is located between about Station P6+462 and Station P6+818. This segment is accessed by an unimproved dirt road oriented in a north-south direction and located adjacent to an evaporation pond. Overhead power lines traverse the existing weir alignment in a north-south direction. Surface features west of the existing weir alignment generally consist of vacant land overgrown with grass and shrubs. Scattered animal burrows were observed throughout the western portion of the levee. Marshland occupies the area east of the existing weir alignment.

Concrete chunks were noted on the surface throughout this segment. Pieces of asphalt concrete, steel piping, discarded carpeting, and wood debris are also present. A rectangular-shaped black plastic tarp is located west of the unimproved access road near Station P 6+650. The tarp was approximately 15 by 20 meters (49.3 by 65.6 feet) and is bordered by sand and cement bags. A wall, approximately 0.3 meter (1 feet) wide by 0.9 meter (3 feet) tall by 18 meter (59 feet) long, constructed of sandbags is located on the northern boundary of the tarp. A second wall of similar dimensions and construction is located approximately 30.5 meter (100 feet) to the south. Both walls are oriented in an east-west direction and run from the evaporation pond to the unimproved access road. Two, large concrete blocks are located at about Station 6+750 along this segment. One block has dimensions of approximately 2.1 meters (6.9 feet) wide by 4.5 meters (14.8 feet) long by 1.5 meters (4.9 feet) tall. The second block measures approximately 1.2 meters (3.9 feet) by 1.2 meters (3.9 feet) by 1.2 meters (3.9 feet).

## 3.2 SUBSURFACE CONDITIONS

### 3.2.1 Field Exploration

The subsurface conditions along the East and West Levees were investigated for the 2000 study by drilling 34 exploratory boring (Borings EB-01 through EB-17 and Borings WB-01 through WB-17) and 28 cone penetration tests (CPT) (EC-01 through EC-13 and WC-01 through WC-15) at the location shown on Figure 2. The results of the previous investigation were presented in Appendix E and Appendix F in the April 2000 report.

For the current study, two (2) additional borings (EB-27 and EB-28) were drilled through the East Levee embankment, and two (2) borings (CB-14 and CB-15) were drilled in the channel. Three (3) additional borings (CB-16, CB-17, and CB-18) were drilled near the proposed West Levee Overflow Weir at locations shown on Figure 7. Please note that borings drilled on the East Levee start with the letter E; similarly borings on the West Levee start with the letter W, and borings within the channel start with the letter C. A detailed discussion of the techniques used for the subsurface investigation is presented in Appendix A. The depths of these borings ranged from 10.2 to 24.8 meters (33.5 to 81.5 feet). Figure A-1 presents the Unified Soil Classification System, as well as guidelines summarizing soil consistency and relative density used in preparation of the boring logs. Figure A-2 is a Boring Log Legend that includes the notation used for the types of samples and methods of advancing them. Comprehensive descriptions of the soils encountered at each location are presented on the Logs of Borings, Figures A-3 through A-10; these include the log of nearby previous Boring EB-01. The soil conditions encountered are summarized in Section 3.2.3 “Soil Conditions”.

### 3.2.2 Laboratory Testing

Soil samples were carefully sealed in the field and returned to our laboratory for testing. Soil classifications made in the field were verified in the laboratory after further examination and testing. Laboratory tests were performed on selected soil samples. These tests include, but are not limited to, water content, dry density and unconfined compressive strength. The results of these tests are presented at the corresponding sample locations on the Log of Borings. Additional tests including Atterberg Limits, sieve analysis, and consolidation were made to better characterize the soil conditions and to estimate the compressibility of the levee fill, foundation material and channel material.

Four (4) samples were selected for consolidation tests based on their quality and soil conditions encountered in the corresponding boring; samples tested include EB-27-5 (depth of about 5.8 m or 19 feet), EB-27-9 (depth of about 13.1 m or 43 feet), CB-17-2 (depth of about 6.5 m or 21 feet) and CB-17-4 (depth of about 9.5 m or 31 feet). Four (4) sieve analyses were conducted to determine the grain size distribution of selected sand and gravel samples. Three (3) Atterberg Limits tests were performed on selected fine-grained soils. Four (4) sieve analyses were conducted to determine the grain size distribution of selected sand and gravel samples.

A more comprehensive discussion of the laboratory testing program is presented in Appendix B.

### 3.2.3 Soil and Groundwater Conditions

We have based our assessment of the soil conditions on two major sources:

- Field exploration conducted for this study (detailed logs of borings are presented in Appendix A);
- Field exploration previously conducted by URS and presented in our April 28, 2000 report; and
- Existing subsurface data from previous investigations by our predecessor firms as well as by others (a list of the sources and boring logs are presented in Table 1).

At the time this report was prepared, surveyed ground surface elevations of borings were not available. Elevations were estimated from nearby cross section geometry provided by the District and from spot elevations provided by CH2MHILL.

For convenience of discussion, the subsurface soil and groundwater conditions along the East Levee and a portion of the West Levee are presented below with reference to the stations of the control line as shown on Figure 2. Furthermore, we previously prepared two color-coded idealized soil profiles illustrating the subsurface conditions encountered in the 1999 and preceding investigations. These profiles are presented in Figure 5 and Figure 6 for the East Levee and West Levee, respectively, in our April 28, 2000 report. Based on those two figures and logs of seven (7) recent borings, we have prepared the following description of the soil and groundwater conditions.

#### *3.2.3.1 East Levee*

#### Station 6+100 to Station 7+050

Levee fill extends to depths ranging from 1.8 to 4.4 meters (6 to 14.5 feet). The fill consists of lean clay with varying amounts of sand and gravel and occasional brick fragments. These levee fills are generally medium to hard in consistency within the upper 1.5 to 2.1 meters (5 to 10 feet) and becomes medium to soft near the fill and foundation soil interface.

Below the levee fill, a deposit of loose to very dense, well-graded to poorly graded sand extends to a depth of 4.9 to 6.1 meters (16 to 20 feet). In Boring EB-28, a medium, sandy clay was encountered below the levee fill to a depth of about 5.3 meters (17.5 feet).

Below the native granular layers, other foundation soils consist of fat and lean clays with varying amounts of sand and organics extending to 13.4 meters (44 feet). The clays are generally soft to stiff in consistency and moderately compressible. Thin seams and lenses of fine sand and gravel were found throughout these clay layers.

Since the rotary wash method was used for the full depth of Borings EB-27 and EB-28, groundwater was not measured. During the 2000 investigation, groundwater was encountered in Boring EB-01 at approximately 4.3 meters (14 feet).

### *3.2.3.2 West Levee Overflow Weir*

**Station 6+500 to 6+800**

A heterogeneous fill consisting of lean clay with miscellaneous debris forms the levee that extends to approximately 4.0 to 4.3 meters (13 to 14 feet). The fill material is very stiff to soft in consistency and contains concrete slabs, brick, asphalt, carpet, and wood debris.

Below the levee fill, a thick deposit of fat clay extends to depths ranging from 10.7 to 14.6 meters (35 to 48 feet). The fat clay is very soft to soft in consistency, highly compressible, and contains varying amounts of organics. Underlying the fat clay is a relatively thin layer of lean clay and silt with occasional gravel. The clay varies in consistency from medium to very stiff, while the silt varies from loose to medium in density.

Beneath the clay and silt layers, granular layers of varying thickness and gradation consisting of poorly and well graded sand to extend to 24.8 meters (81.5 feet). These granular layers are medium dense to very dense in relative density.

Groundwater level measured in Boring CB-18 was approximately 3.7 meters (12 feet).

### *3.2.3.3 River Channel*

Two Borings, CB-14 and CB-15, were drilled in the river channel, i.e. between the inboard toes of the East Levee and West Levee for Task 2.4.6. Fill was encountered to depths of 0.9 meter (3.0 feet) in Boring CB-14 and 0.8 meter (2.5 feet) in Boring CB-15; it ranges from dense clayey sand with gravel to soft lean clay with gravel. In Boring CB-14, the fill was underlain by stiff lean clay and soft to medium fat clay extending to a depth of about 7.6 meters (25 feet). However, a clayey to silty gravel layer was encountered within the lean and fat clay layers from a depth of 2.1 to 4.0 meters (7 to 13 feet). A loose silty sand layer underlies the clay layers to a depth of 10.2 meters (33.5 feet). In Boring CB-15, fat clay underlies the fill and is soft to medium in consistency. From depths 3.2 to 7.6 meters (10.5 to 25 feet) the fat clay was noted to be sandy and contains thin fine to medium sand lenses.

Groundwater level measured in Boring CB-15 was at approximately 1.8 meters (6 feet).

# SECTION 4

## 4.1 CROSS SECTIONS SELECTED FOR ANALYSIS

During our meeting of November 10, 1998 with the District, we presented two color-coded subsurface profiles (Figures 6 and 7 in the April 2000 report), one for the East Levee and a second for the West Levee. Our interpretation of the soil layers in each profile was based on our evaluation of the subsurface information, including both borings and cone penetration tests (CPTs). Based on these two profiles, we identified a total of 23 levee sections as candidates for analysis along the entire alignment between Station 6+200 and Station 17+500; the levee sections are listed in Table 3. At the request of the District for the 2000 study, we selected and performed engineering analyses for what we believe represented the seven (7) most critical cross sections within the 23 levee sections.

For this current study for Task 2.4.6, we have analyzed two (2) additional cross sections along the East Levee between Stations E 20+201 and E 20+985. A list of the two (2) cross sections is presented in Table 4 and the location of each cross section in plan view is shown in Figure 2. The basis for selection of the two (2) cross sections is presented below.

For the overflow weir, we elected not to perform site specific stability analysis because the side slopes are relatively flat (4H to 1V). It is our engineering opinion that the levees at this location will meet the Corp's stability criteria.

### 4.1.1 Selection of 23 Levee Cross Sections

The two idealized soil profiles (Figure 6 and Figure 7 of the April 28, 2000 report) presented in our November 10, 1998 meeting were based on the logs of 185 previous borings, as well as the 34 new borings and 28 new CPTs for the 2000 study. Based on our engineering judgment and interpretation of these two profiles, we identified 13 levee sections along the west levee (Sections W1 through W13), and 10 levee sections along the east levee (Sections E1 through E10). The approximate channel control line station limits of each levee section are listed in Table 3. We also plotted the limits of each levee section in plan view on Figure 2. Although a total of 23 levee sections for both West and East Levees were initially identified, in our opinion three (3) sections of the West Levee (Sections W2, W9 and W10) have similar subsurface conditions as the East Levee (Sections E2, E9 and E10, respectively). Therefore, from an engineering viewpoint, we believe the subsurface conditions along the entire project alignment can be represented by a reduced total of 20 levee sections. One of the 23 previously identified levee sections lie within the limits of this study.

### 4.1.2 Selection of Two Levee Cross Sections for Task 2.4.6

According to our Agreement, a maximum of two (2) levee cross sections were to be analyzed. We have selected two (2) sections that we believe are reasonably representative of the most critical conditions. Critical is defined as most likely having the lowest factor of safety for slope stability, based on our engineering judgment. The two (2) levee cross sections we selected are shown in Table 4 and are defined as Cross Sections 31 and 32. The reasons for selecting these two (2) are listed under the column entitled "comments" in Table 4; they vary from the presence of soft, fat clay (CH), to a depressed maintenance road, to a levee raise. The location of each cross section is shown in plan view in Figure 2.

Initially, one critical cross section was chosen from each original levee section identified with similar soil conditions. For comparison, the average side slope inclinations and slope heights were tabulated. Selection of the critical cross section in each levee section was based on slope inclination and slope height. Slopes with steeper side slopes or larger heights were judged to have a lower factor of safety. Additional cross sections were then selected where deemed necessary by project scope or engineering judgment.

The levee improvements (as indicated by the plans prepared by CH2MHILL, dated April 2002) were incorporated into the selected critical cross sections. The proposed geometry, including levee raise and excavation beyond the inboard toe, was incorporated into the slope stability and seepage analyses. The design high water level was obtained for each cross section from the plans prepared by CH2MHILL.

Soil conditions were also a factor in the selection of the critical cross sections. Soft clay and loose sand layers are considered critical. The presence of soft, fat clay (CH) in the levee foundation was identified using idealized soil profiles previously presented as Figures 6 and 7 of the 2000 report.

We prepared two drawings for each of the two (2) cross sections: (1) one drawing showing a cross section representing the soil conditions and existing levee geometry, and a table summarizing the selected geotechnical design parameters (Figures 9 and 10); and (2) a second drawing showing four graphs where the undrained shear strength ($S_u$), moisture content, dry density and blows per foot are plotted with respect to elevation (Figures 11 and 12). The latter drawing includes data from all available borings in both levees within the station limits of the relevant levee section. The District furnished all original geometric levee sections; levee improvements were then incorporated based on the plans prepared by CH2MHILL.

### 4.1.3 Selection of Shear Strength Parameters

At the two levee cross sections, shear strength parameters were selected for each soil layer based on the field and laboratory data. The selected design parameters are summarized in the upper left-hand corner or upper right-hand corner table of Figures 9 and 10 for Cross Sections 31 and 32, respectively. These methods are described in more detail as follows.

#### *4.1.3.1 Cohesive Soils in Levee and Levee Foundation*

**Undrained Strength**

As an example, refer to Cross Section 31 on Figures 9 and 11 where four cohesive soil layers are shown in the levee fill and foundation including:

- Layer 1 is Lean Clay fill (CL), about 2.1 meters (7 feet) thick
- Layer 2 is Lean Clay (CL), about 3.8 meters (12.5 feet) thick

- Layer 3 is Fat Clay (CH), about 4.7 meters (15.5 feet) thick
- Layer 5 is Lean Clay (CL), about 1.8 meters (6 feet) thick

As shown in $S_u$ versus elevation in Figure 11, all of the unconfined compression, undrained unconsolidated triaxial shear strength tests (UU) and CPT tests results within the limits of Section E1 are plotted. Based on this plot for Layer 1 (Elevation 4.0 to 1.8 meters or Elevation 13 to 6± feet) an $S_u$ value of 71.8 kPa (kilo Pascals) [1,500 pounds per square foot (psf)] was selected as represented by the solid vertical line. For Layer 2 (Elevation 1.8 to –2.0 meters or Elevation 6 to –6.5± feet) an $S_u$ value of 35.9 kPa (750 psf) was selected. For Layer 3 (Elevation –2.0 to –6.7 meters or Elevation –6.5 to -22± feet) an $S_u$ value of 43.1 kPa (900 psf) was selected. For Layer 5 (Elevation –7.9 to –9.8 meters or Elevation -26 to -32± feet) an $S_u$ value of 47.9 kPa (1,000 psf) was selected. These values were then entered in the table at the upper left-hand corner of Figure 9. It should be noted that the $S_u$ value selected is not a lower bound; in some cases the $S_u$ values plotted are believed to underestimate the soil strength because of the presence of sand interbeds in the samples tested.

In a similar manner, the moisture content and dry density for each layer in Cross Section 31 were obtained from the other plots in Figure 11 and entered into the table in the upper right hand corner of Figure 9.

The embankment strengths were reduced to account for progressive failure in embankments on soft clay foundations. Since the embankment (fill) shear strengths were considered to be relatively high, the embankment strengths were reduced according to the procedures outlined by Duncan and Buchignani (1975). The plot used to determine the embankment strength reduction factor ($R_E$) is presented in Figure 13. For the foundation, we used a foundation strength reduction factor of 1.0 for Type A soil, also presented in Figure 13.

### Effective Strength

The data from consolidated undrained (CU) triaxial shear strength tests with pore pressure measurements are presented in Table 5. These test results include both existing tests and new tests performed for this current investigation; they are divided into the following five categories:

- Lean clay fill (CL), dry density < 1,762 kilograms per cubic meter (kcm) [110 pounds per cubic foot (pcf)
- Lean clay fill (CL), dry density > 1,762 kcm (110 pcf)
- Fat clay (CH), dry density < 1,281 kcm (80 pcf)
- Lean clay (CL), 1,281 kcm (80 pcf) < dry density < 1,602 kcm (100 pcf)
- Lean clay (CL), 1,602 kcm (100 pcf) < dry density < 1,762 kcm (110 pcf)

Two effective strength parameters are shown in Table 5 including $\phi'$ (angle of internal friction) and c' (cohesion); values are listed for peak stress, 10 percent and 20 percent strain criteria. A design value was selected between the peak stress and 10 percent strain; in our judgment 20 percent strain is too large a deformation for the levee to undergo. The value from Table 5 was then entered into the upper right-hand corner table of Figure 9 for Layers 1 through 3 and 5. For example, Layer 1 was lean clay fill with a dry density of 1,602 kcm (100 pcf); from Table 5 for a dry density less than 1,762 kcm (110 pcf), $\phi'$ = 24 degrees and c' = 21.5 kPa (450 psf) were selected.

Table 5 was prepared based on the graphical CU test results plotted in Figures 14 through 18. For example for the lean clay fill category with samples with dry density less than 1,762 kcm (110 pcf) for peak stress, all q versus p values were plotted in Figure 14. The q and p values are defined as follows:

- $q = \frac{1}{2}(\sigma_1' - \sigma_3')$
- $p = \frac{1}{2}(\sigma_1' + \sigma_3')$

The best fit straight line (method of least squares) was then plotted in Figure 14 for peak stress and the $\phi'$ and c' values calculated (as shown in that figure) as 23.7 degrees and 21.8 kPa (456 psf), respectively.

### *4.1.3.2 Cohesive Soil in Channel*

#### Undrained Strength

Borings were advanced in the channel (beyond the inboard toe of the levee) as part of this study in order to assess the strength of the materials subjected to periods of wetting and riverbed deposition. In general, the undrained strengths of the channel sediments were lower than undrained strengths of soils within the levee or levee foundation.

The extent of the channel sediments was estimated by extending a ½:1 (horizontal to vertical) line beneath the inboard levee toe. Cohesive soil inboard of this line was assumed to have channel sediment properties. Cohesive soil outboard of this line was assumed to have the properties determined from the index property plots of levee and foundation soils.

For Cross Sections 31 and 32, the dry density, moisture content, and undrained strength of the channel sediments were estimated from the results of dry density, moisture content, and unconfined compression tests performed on samples from Borings CB-14 through CB-18. The plots of undrained strength, moisture content and dry density versus depth for the channel sediments are presented in Figures 11 and 12. Selected design parameters for the channel sediments are represented by a dashed vertical line and the values were inserted in the tables located in Figures 9 and 10. The selected undrained strength was estimated using the results of laboratory testing, local experience, and engineering judgment.

## Effective Strength

The effective strengths of the cohesive channel sediments were determined in the same manner as outlined for cohesive soils in the levee and levee foundation.

### *4.1.3.3 Granular Soils*

Effective strength parameters were estimated by the following method (refer to Layer 4, silty sand, in the upper right-hand corner table of Figure 9). The effective stress $\sigma_v'$ at the middle of Layer 4 was calculated to be about 105.3 kPa (2,200 psf). For a Standard Penetration Test N value of 7 and $\sigma_v'$ of 105.3 kPa (2,200 psf), the estimated relative density ($D_r$) is about 50 percent (see Figure 19). According to a published graph (see Figure 20) by NAVFAC (1982) for a silty sand and $D_r$ of 50 percent, and an approximate dry unit weight of 105 pounds per cubic foot (pcf), $\phi'$ is about 33 degrees.

In conclusion, it is our opinion the two sections selected and listed in Table 4 (Cross Sections 31 and 32) are reasonably representative of the subsurface conditions that may be critical from a levee stability and design viewpoint. We performed slope stability analyses at each of these sections based on the parameters summarized in Figures 9 and 10 for the existing levee geometry with the improvements as specified by CH2MHILL.

## 4.2 METHOD OF ANALYSIS

Four design conditions were analyzed for the slope stability evaluation in accordance with the Corps Manual for the Design and Construction of Levees (EM 1110-2-1913). The conditions, the slopes analyzed, shear strength guidelines, and minimum factors of safety required for design are listed below.

| Case | Design Condition | Slope Analyzed | Required Minimum Factor of Safety |
|---|---|---|---|
| I | End of Construction | Riverside and Landside | 1.3 |
| II | Sudden Drawdown | Riverside | 1.0 |
| III | Steady Seepage from Full flood Stage | Riverside and Landside | 1.4 |
| IV | Earthquake | Riverside and Landside | Not applicable |

As outlined in the Corps' guidelines, earthquake loadings are not considered in analyzing the stability of levees because of the low probability of an earthquake coinciding with a period of high water. Nevertheless, we evaluated the seismic stability of the East Levee in the following manner because of the close proximity to major fault systems such as the San Anderas and Hayward:

- Performed pseudo-static slope stability analysis using a seismic coefficient approximately one-half of the peak ground acceleration (0.3 to 0.33); this is consistent with the California Department of Conservation, Division of Mines and Geology, Special Publication 117, "Guidelines for Evaluating and Mitigating Seismic Hazards in California," (1997).

- Performed simplified deformation analysis for cross-sections exhibiting a pseudo-static slope stability safety factor less than 1.0 (Makdisi and Seed, 1977) to estimate the magnitude of potential permanent deformation due to strong earthquake shaking.

The stability of the levees under Cases I, II, III and IV (pseudo-static) was evaluated using a limit equilibrium method based on Spencer's procedure of slices as coded in the program SLOPE/W (Geo-Slope, Int., 1992). In Spencer's procedure, all forces are assumed to have the same inclination, and all requirements for static equilibrium are satisfied. The trial and error solution involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium are satisfied. In addition, the slope stability results of some cross sections computed by SLOPE/W also were evaluated using a simplified chart method. The failure surfaces identified by SLOPE/W using Spencer's procedure are circular.

### 4.2.1 Pore Pressures

The SEEP/W program (Geo-Slope International, 1992) was used to estimate the phreatic surface in the levee for the critical flood stage under the steady seepage condition based on the design water levels provided in the plans prepared by CH2MHILL. The SEEP/W program and the seepage analyses are described in further detail in Section 6 "Seepage Evaluation" of this report. For a given cross section and design condition, the wetted face (phreatic surface) was estimated. The phreatic surface generated by SEEP/W was imported into SLOPE/W to calculate the pore pressures within the levee for Cases II and III; it represents the worst-case groundwater condition.

### 4.2.2 Static Loading Conditions

The static loading conditions for slope stability analysis prescribed by the United States Army Corps of Engineers (Corps) are listed below.

- End of Construction
- Sudden Drawdown
- Steady Seepage from Full Flood Stage

A discussion of the conditions for analysis and their respective strength parameters is found in the following sections.

#### *4.2.2.1 End of Construction*

The end of construction case represents the condition immediately following construction when excess pore pressures have been induced by the construction, and not enough time has passed for the pressures to dissipate. For this reason, undrained conditions exist for an impervious embankment and foundation soils. Strength results from laboratory unconsolidated-undrained tests were used for the fine grained soils. The effective friction angle was used for the granular soils, as they are assumed to be pervious and free draining.

#### 4.2.2.2 Sudden Drawdown

The sudden drawdown case represents the condition whereby a prolonged flood stage saturates at least the major part of the upstream (riverside) embankment portion and then drops faster than the soil can drain. This causes the development of excess pore pressures that may result in the riverside slope becoming unstable.

For this analysis, a composite envelope was used for the design strengths for impervious soils. The strength envelope consisted of the drained strengths until the intersection of the undrained strength was reached. At that point and beyond, the undrained strength was used. These strengths are in accordance with the Corps Manual.

#### 4.2.2.3 Steady Seepage From Full Flood Stage

The steady seepage case represents the condition where the water remains at or near full flood stage so that the embankment becomes fully saturated and a condition of steady seepage occurs. The design shear strength should be based on the effective stresses; this should correspond to the residual strengths where previous shear deformation or sliding has occurred.

For this analysis, the effective stress envelope is based on consolidated undrained test results. Recognizing the non-linear nature of the shear strength envelope for higher effective stresses, we have modified the strength using a composite envelope. Drained strengths were used until they became greater than the undrained strengths. At this point and beyond, an envelope midway between the drained and undrained strengths was used. The water levels used are at the design high levels as specified on the plans prepared by CH2MHILL.

### 4.2.3 Earthquake Loading Condition

The earthquake case represents the condition when the levee is subjected to earthquake motion. As discussed above, we performed a 2-step evaluation for the earthquake loading. In the first step, we selected a seismic coefficient equal to one-half the peak horizontal ground acceleration as outlined in the 1997 Guidelines for Evaluating and Mitigating Seismic Hazards in California. The peak ground acceleration was determined using a deterministic approach where no consideration is given to the probability of an earthquake occurring. The maximum earthquake magnitudes adopted by the Working Group on Northern California Earthquake Potential (1996) were used for each of the seismic sources. Empirical attenuation relationships were used to calculate the 84th percentile horizontal ground accelerations as discussed previously in Section 2.3 "Ground Motions". Accelerations were determined for four segments along the reach of the Lower Guadalupe River, as presented in Table 2a through 2d. For the portion of the alignment analyzed in this study (Task 2.4.6), the design seismic coefficient is 0.55 g for Cross Sections 31 and 32. The strength parameters used for the earthquake condition were the same as for the end of construction. In the second step, we estimated the permanent deformation at cross-sections with safety factor less than 1.1 under pseudo-static loading analyzed in the first step.

## 4.3 STABILITY RESULTS OF LEVEE SECTIONS

The minimum (critical) factors of safety computed using the various methods of analysis are summarized in the table below.

| Case | Design Condition | Factor of Safety | | |
|---|---|---|---|---|
| | | Minimum Required | Cross Section 31 Minimum | Cross Section 32 Minimum |
| I | End of Construction (Riverside) | 1.3 | 2.8 | 2.3 |
| I | End of Construction (Landside) | 1.3 | 3.1 | 5.3 |
| II | Sudden Drawdown (Riverside) | 1.0 | 2.2 | 2.2 |
| III | Steady Seepage from Full Flood Stage (Riverside) | 1.4 | 4.4 | 3.7 |
| III | Steady Seepage from Full Flood Stage (Landside) | 1.4 | 2.1 | 3.9 |
| IV | Pseudo-Static Earthquake (Riverside) | 1.0 | 1.0 | 1.1 |
| IV | Pseudo-Static Earthquake (Landside) | 1.0 | 1.3 | 2.3 |

The approximate locations of the critical slip surfaces are shown on Figures 9 and 10. The analyzed sections appear to be stable for each of the design conditions.

For a pseudo-static safety factor less than 1.1, we estimated the permanent deformation that could occur under the maximum credible earthquakes. The following table presents a summary of our estimates:

| Cross-Section Number | Yield Acceleration (g) | Estimated Average Total Deformation (mm) | Estimated Average Vertical Deformation (mm) |
|---|---|---|---|
| 31 | 0.65 | 0 | 0 |
| 32 | 0.17 | 405 | 284 |

The table above presents results in terms of total deformation ($\Delta_{\text{total}}$) along the potential sliding surface and estimated vertical displacement ($\Delta_{\text{vertical}}$). For the potential sliding surface through or near the crest of the embankment, $\Delta_{\text{vertical}} = 0.7\ \Delta_{\text{total}}$ was estimated based on the slope of the potential failure surface at the crest. As shown, we estimate vertical deformation to be nil at Section 31 and 284 mm (11 inches) at Section 32. At these locations we anticipate that shaking from a major earthquake will result in some cracking of the levee, requiring regrading of the levee.

# SECTION 5

## 5.1 SUBSIDENCE

From 1916 to 1966 in the San Jose area of Santa Clara Valley, the artesian head declined from 55 to 73 meters (180 to 240 feet) in response to groundwater withdrawal. As a direct result of the artesian head decline, the land surface subsided as much as 3.9 meters (12.7 feet) in San Jose (Poland and Ireland, 1985). The recovery of artesian head since 1967 has been substantial. In downtown San Jose, the artesian head recovered 21.3 to 30.5 meters (70 to 100 feet) in the 8 years to 1975. The subsidence appears to have been stopped by raising the artesian head in the aquifers. From 1934 to 1960, maximum subsidence of 1.5 meters (5 feet) occurred in San Jose and Sunnyvale. From 1960 to 1967, maximum subsidence of 1.1 meters (3-1/2 feet) occurred in San Jose and Santa Clara; the average rate of subsidence was 152 mm (1/2 foot) per year, the most rapid rate of subsidence in any 7-year period. Based on the land subsidence contour map prepared by United States Geological Survey (Poland and Ireland, 1967), the segment of Guadalupe River within the scope of study experienced subsidence ranging from 1.8 to 2.4 meters (6 to 8 feet) between the period of 1934 and 1967. Since 1967, subsidence has been significantly reduced. In San Jose Wells 7S/1E-16C6 and C11, an extensometer recorded compression decreasing from 305 mm (1 foot) in 1961 and 1962 to 76 mm (1/4 foot) in 1967 and to 3 mm (1/100) foot in 1973. Based on the referenced contour map, a maximum drop of 1.1 meters (3½ feet) in elevation of the levees since their improvement construction in 1960 probably can be attributed to land subsidence.

## 5.2 LEVEE SETTLEMENT

### 5.2.1 Inferred Settlement

Based on review of the (1) as-built drawings for the 1960 and 1963 improvement construction of the levees and (2) those for the 1985 and 1995 levee raising and widening of the Guadalupe River channel, we have estimated the levee settlement between Station 7+008 and Station 14+500. Estimates of the settlement are inferred because they are primarily based on the change in elevation of the crest of the levees. No settlement monitoring was performed. Our review indicates that the lowering of crest elevations occurred mostly in areas north of Station 12+000 (Montague Expressway). The levee portions south of Station 12+000 appear to have experienced little or no change in crest elevation. We estimate that the portions of levees north of Station 12+000 settled due to consolidation approximately 229 to 762 mm (average of 381 mm) or ¾ to 2½ feet (average of 1¼ feet) for the West Levee and 396 to 1189 mm (average of 933 mm) or 1¼ to 4 feet (average of 3 feet) for the East Levee for the period of 1960 to 1983. These inferred consolidation settlements deduct the groundwater withdrawal related land subsidence settlement estimated from 1960 to 1967. North of Station 12 +000, we estimated the subsidence settlement to be 457 to 762 mm (1½ to 2½ feet) during this 7-year period.

### 5.2.2 Predicted Settlement

These estimates have been compared to settlement calculations based on consolidation test results from a 1960 and 1999 studies and our current investigation (2002). It is assumed that the height of the fill at each stage of construction was added instantaneously. More specifically, the construction stages are presented in the following table.

| Cross Section | Station | Levee | Fill Thickness (feet) | | | |
|---|---|---|---|---|---|---|
| | | | 1963 | 1983 | 1995 | 2003 |
| 31 | 6+300 | East | 8.5 | 4.0 | 0 | 2.5 |
| 32 | 6+780 | East | 10.5 | 2.0 | 0 | 4.0 |

The locations of the cross sections are shown in plan view in Figure 2, Site and Field Exploration Location Plan. These cross sections are the same cross sections used in the levee slope stability analysis described in Section 4 "Levee Slope Stability Evaluation".

The time rate of settlement for the two (2) cross sections are presented graphically on Figures 21 and 22 for the period from 1963 to 2010. The following table summarizes the estimated consolidation settlements from 1963 to 2003.

| Cross Section | Estimated Consolidation Settlement | | | | | |
|---|---|---|---|---|---|---|
| | From 1963 to 1983 | | From 1963 to 1995 | | From 1963 to 2003 | |
| | mm | inches | mm | inches | mm | inches |
| 31 | 679 | 26¾ | 893 | 35¼ | 894 | 39 |
| 32 | 735 | 29 | 984 | 38¾ | 989 | 39 |

Cross Sections 31 and 32 are located north of Station 12+000 (near Montague Expressway) and positioned on the East Levee. The values shown in the above table are consistent with those estimated (inferred) from the As-built drawings. For example, the range of estimated consolidation settlement from the above table for the East Levee for the period 1960 to 1983 ranged from 679 mm (Cross Section 31) to 735 mm (Cross Section *32*). As described in Section 5.2.1 "Inferred Settlement" the range of inferred consolidation settlement for the East Levee was 396 to 1,189 mm. Therefore, we have concluded the approach and methodology used in our settlement analysis are reasonable for the East Levee.

## 5.3 FUTURE SETTLEMENT OF EAST LEVEE

Between the Alviso Marina (Approximately Station 6+130) and the UPRR Bridge (Station 7+008), the East Levee will be raised a maximum of 0.8 meter (2.5 feet).

Most of the East Levee will undergo settlement in the future from ongoing consolidation of the foundation clay soil due to fills placed in 1963, 1983 and presumably new fill placed in 2003. Consequently, we have estimated the following settlement due to long term consolidation.

| Cross Section | Estimated Future Settlement | |
|---|---|---|
| | mm | inches |
| 31 | 98 | 3¾ |
| 32 | 168 | 6½ |

The future settlement along the East (Alviso) Levee north of the UPRR Bridge is estimated to be on the order of 98 to 168 mm (3¾ to 6½ inches) for the levee due to continuing long term consolidation and the addition of new fill presumed to be placed in 2003.

## 5.4 WEST LEVEE OVERFLOW WEIR

We understand that maximum new fill heights of approximately 1 meter (3.3 feet) are planned for the proposed West Levee Overflow Weir. Therefore we have estimated the long term consolidation settlement due to placement of this new fill (1 meter). The estimated additional settlement and rates, including the effect of surcharging each area with 1.5 or 3.0 meters (5 or 10 feet) of temporary fill (surcharging pressure of approximately 31 and 62 kPa, respectively), are shown in the following table. Note these values also include the settlement from ongoing consolidation of the foundation clay due to fills placed previously at this site (assumed to be 12 feet of fill placed in 1983).

| Wall | Station | Max. New Fill Height (m) | Estimated Time Rate Settlement (mm) | | | | Estimated Ultimate Settlement from New Fill in 2003 (mm) | 90% Consol - idation (mm) | Surcharge? ($\rho_{90\%}$ > 25.4 mm) (1 - inch) | Time to 90% Consolidation (months) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1 month | 3 months | 6 months | 1 year | | | | No Surcharge | w/1.5 m Surcharge | w/3.0 m Surcharge |
| West Levee Overflow Weir | P6+517 to P6+763 | 1 | 47 | 74 | 98 | 132 | 272 | 245 | Yes | 60 | 12 | 5 |

As shown above, the time to achieve 90% consolidation is about 60 months. If a surcharge height of 1.5 meters (5 feet) is placed for a period of 12 months or 3.0 meters (10 feet) for a period of 5 months, 90% consolidation settlement could be achieved. If surcharge is selected, we recommend that settlement plates/devices be installed prior to placement of fill and settlement be monitored before, during and following fill placement activities.

It should be noted that the existing fill material located beneath the proposed weir appears to have been placed loosely and also contains miscellaneous debris, concrete chunks, loose brick, and wood debris. This material was noted on the ground surface as well. Due to the variable nature of the existing fill (soft, compressible zones, voids, and/or materials), it is expected that the estimated consolidation settlement may exceed the values shown in the above table. It is difficult to estimate the settlement due to compression of the existing fill; it could be higher by a factor of ½. Therefore, the estimated ultimate settlement could be on the order of 408 mm (1.5 x 272 mm).

In order to mitigate this variable settlement the existing fill may be removed and replaced with select engineered fill. The following table presents estimated long term consolidation settlement due to removal and replacement of existing fill:

| Wall | Station | Max. New Fill Height (m) | Estimated Time Rate Settlement (mm) | | | | Estimated Ultimate Settlement from New Fill in 2003 (mm) | 90% Consol - idation (mm) | Surcharge? ($\rho_{90\%} > 25.4$ mm) (1 - inch) | Time to 90% Consolidation (months) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1 month | 3 months | 6 months | 1 year | | | | No Surcharge | w/1.5 m Surcharge | w/3.0 m Surcharge |
| West Levee Overflow Weir | P6+517 to P6+763 | 1.8 | 72 | 117 | 156 | 208 | 395 | 355 | Yes | 61 | 18 | 9 |

Since the new select engineered fill is more dense (heavier) than the existing miscellaneous fill, the estimated ultimate settlement of 395 mm is greater than the 272 mm.

As shown above, the time to achieve 90% consolidation is about 61 months. If a surcharge height of 1.5 meters (5 feet) is placed for a period of about 18 months, or 3.0 meters (10 feet) for a period of 9 months, 90% consolidation settlement could be achieved. If surcharge is selected, we recommend that settlement plates/devices be installed prior to placement of fill and settlement be monitored before, during and following fill placement activities.

# SECTION 6

## 6.1 METHOD OF ANALYSIS

The flood levels used in the analysis of each of the selected sections were provided by CH2MHILL on the plans dated August 2001 and April 2002. The flood levels are summarized as follows:

| Cross Section Number | Station (meters) | 100-year Flood Elevation (meters, NAVD)* | (meters, NGVD)** | (feet, NGVD) |
|---|---|---|---|---|
| 31 | E 6+300 | 4.10 | 3.3 | 11.0 |
| 32 | E 6+780 | 4.40 | 5.3 | 12.0 |

* From August 2001 Drawings, CH2MHILL
** Converted to NGVD Datum

Seepage through the levee during maximum flood was analyzed using a steady state analysis. A finite element program, SEEP/W [developed by Geo-Slope International (1992)], was used to perform calculations. SEEP/W is a computer program of two-dimensional code with isoparametric and higher order finite elements capable of solving saturated and unsaturated flow conditions using steady state and transient analysis. The program can represent multiple soil types with anisotropic hydraulic conductivity; it can also represent prescribed constant and transient boundary conditions.

The two boundary conditions established for the steady state analysis are constant head and no flow conditions. The nodes beneath the water on the inboard side of the levee used the boundary condition of head equal to the height of the design water level. A boundary condition of no flow was established across the bottom of the finite element mesh. The nodes on the upstream portion of the levee were assigned a flux-type review flow boundary condition. In this case, after the heads are computed for all nodes, the review nodes are modified if any one of the review nodes has a computed head greater than the y-coordinate of the node (that is, if the porewater pressure is greater than zero). Using the maximum pressure review option, the review node with the maximum positive pressure head is identified by SEEP/W. The head at this node is then set to the y-coordinate for the next iteration. At the outboard boundary of the mesh, a constant head boundary was established. The constant head value was inferred from water level observed in historical borings in the vicinity of the cross section.

## 6.2 CROSS SECTIONS AND RESULTS OF ANALYSIS

In general the levees are constructed of well compacted lean clays or lean to fat clays. Furthermore, most of the native soils directly underlying the levee fills are lean to fat clays. Since the clayey soils are relatively impervious, it is our engineering opinion that generally (1) seepage through and underneath the levees during the design flood will be relatively small and (2) the potential for piping at the outboard toe will also be very low. To confirm these opinions, we performed a steady-state seepage analysis on the 2 critical cross sections. These cross sections correspond to the cross sections listed and described in Section 4.1 “Cross Sections Selected for Analysis.”

The coefficients of hydraulic conductivity (permeability) for three types of soil (lean clay, silty sand and sand) were selected from the following laboratory and published data. They included laboratory permeability test results of relatively undisturbed and remolded samples, broad empirical relationships between soil types and permeability (Cedergren, 1967) as shown in Figure 23, and correlation relationships based on grain size distribution as described in the EM-1110-2-1913 (COE, 1978) and the NAVAC DM-7.1 (1982). Permeability tests were performed for the 2000 study on three relatively undisturbed soil samples as reported in Appendix G of that report. The coefficient of hydraulic conductivity (permeability), k, determined from these samples are as follows:

| Sample Number | Depth (feet) | Soil Type | Location | Dry Density (pcf) | k (cm/second) |
|---|---|---|---|---|---|
| EB 03-16-4 | 30 – 31.5 | Lean Clay | Foundation | 101.5 | $3.3 \times 10^{-7}$ |
| EB 11-9-4 | 14.5 – 16 | Silty Sand | Foundation | 90.3 | $5.1 \times 10^{-4}$ |
| WB 10-11-4 | 26 – 27.5 | Silty Sand | Foundation | 125.2 | $1.2 \times 10^{-4}$ |

The values assigned to the corresponding soil types are presented in the following table. The ratio of anisotropy of the horizontal to vertical conductivity was assumed to be 4 for all soil types.

| Soil Type | Coefficient of Hydraulic Conductivity ($k_h$) | | Selected Values | |
|---|---|---|---|---|
| | cm/sec | ft / hr | cm/sec | ft / hr |
| Sand | $10^{-2}$ to $10^{-4}$ | 1.2 to 0.012 | $10^{-2}$ | 1.2 |
| Silty Sand, Clayey Sand | $5\times10^{-3}$ to $5\times10^{-4}$ | 0.7 to 0.07 | $6 \times 10^{-3}$ | 0.7 |
| Lean to Fat Clay | $10^{-5}$ to $10^{-6}$ | $1.2\times10^{-3}$ to $1.2\times10^{-4}$ | $10^{-5}$ | $1.2 \times 10^{-3}$ |

The steady-state phreatic surface line is presented on Figures 9 and 10 for Cross Sections 31 and 32, respectively. Two representative or critical cross sections were selected. The SEEP/W output for the steady state condition at these cross sections is presented in Figures 24 and 25 for Cross Sections 31 and 32, respectively. These figures also present the estimated flow quantities in cubic feet per hour per linear foot for a steady state seepage condition with river levels at 100-year elevations, as summarized below. In particular, the seepage analysis we performed for Cross Section 32 assumes the maintenance road fill is low permeability material consisting of lean clay.

| | | | | Estimated Seepage Quantity | | |
|---|---|---|---|---|---|---|
| Figure Number | Cross Section Number | Levee | Station | Cubic Feet per Hour per Linear Foot | Gallons per Minute per Linear Foot | Comment |
| 24 | 31 | East | 6+300 | 7.083E-08 | 8.831E-09 | |
| 25 | 32 | East | 6+780 | 8.983E-08 | 1.120E-09 | |

For a 30.5-meter (100-foot) long section, the total flow quantity through the levee at each of the two sections is expected to be less than 1/10 gallon per minute (gpm). In our opinion these seepage quantities are very small. However, it should be emphasized that the quantity of estimated seepage refers to the underground water flow through a theoretical cross-section at the toe, i.e. no seepage exiting the toe of the levee. It is our opinion that piping or similar type of hydraulic failure at the toe is unlikely for the levees.

# SECTION 7

## 7.1 BACKGROUND

As noted in the plans and July 2002 Design Drawings prepared by CH2MHILL, the levee improvements between Station 6+130 and Station 7+008 include the construction of two reinforced concrete floodwalls on the East Levee. The floodwalls vary in height (above finished grade) from 0.25 to 1.35 meters (0.8 to 4.4 feet). The vertical floodwall will be positioned with the vertical face matching the inboard hinge point of the crest; this will result in access to a full width maintenance road even during the 100-year design flood.

We provided preliminary design parameters for the floodwalls in our April 14, 2000 and October 23, 2001 letters to the District and to CH2MHILL, respectively. A copy of each letter is presented in Appendix C. These parameters are defined by reach as designated by CH2MHILL.

## 7.2 CORPS REQUIREMENTS

We have reviewed (1) our preliminary design parameters in the two letters listed in Appendix C and (2) the CH2MHILL Map and General Plan drawings dated August, 2001. At that time, no specific design system was utilized, other than some Caltrans criteria. We now understand the Corps criteria will be used. Therefore, we have revised our recommendations consistent with the Corps Manual No. EM 1110-2-1913 (Design and Construction of Levees, April 30, 2000) and Corps Manual No. EM 1110-2-2502 (Engineering and Design, Retaining and Flood Walls, September 29, 1989). According to EM 1110-2-1913, levee enlargements are constructed either by adding additional earth fill or by constructing a floodwall, "I"-type or "inverted T"-type, on the crown.

The I floodwall is a vertical wall partially embedded in the levee crown. The stability of such walls depends upon the development of passive resistance from the soil. For stability reasons, I floodwalls rarely exceed 2.13 meters (7 feet) above the ground surface, according to the Corps. One common method of constructing an I floodwall is by combining sheet piling that is driven into the levee embankment, and the upper part is a reinforced concrete section capping the steel sheet piling.

An inverted T floodwall is a reinforced concrete wall whose members act as wide cantilever beams in resisting hydrostatic-pressures acting against the wall. According to the Corps, the inverted T floodwall is used to make floodwall levee enlargements when walls higher than 2.13 meters (7 feet) are required.

As described in the 30-Percent Design Submittal, CH2MHILL analyzed the following three wall types.

- Cantilever L-type wall on spread footing (refer to Figure A-2 in the 30-Percent Design Submittal);
- Cantilever inverted T-type wall on spread footing (refer to Figure A-3 in the 30-Percent Design Submittal); and
- Cantilever I-type wall on 600-mm-diameter (24 inches) cast-in-drilled-hole (CIDH) pile with a pile cap (refer to Figure A-4 in the 30-Percent Design Submittal).

Based on their analysis, CH2MHILL recommended that the cantilever L-type wall with spread footing be constructed because of its low initial costs and simpler construction methods. However, we understand the shear key has been moved to coincide with the riverside edge of the footing.

## 7.3 WALL SETTLEMENT

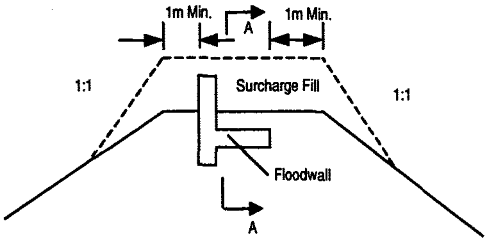
As described in Section 5.3 "Future Settlement of East Levee", we have estimated future total settlement of the existing East Levee at floodwall FW6E to be on the order of 98 mm (3¾ inches). At floodwall FW1E, we have estimated future total settlement of the existing East Levee to be on the order of 168 mm (6½ inches). This settlement is the result of ongoing consolidation settlement of the foundation clay soil due to fills placed in 1963, 1983 and presumably in 2003.

We have also estimated additional settlement at FW6E due to new fill to be placed behind the floodwall ranging in height no more than 0.7 meter (2.5 feet). The estimated additional settlement and rates, including the effect of surcharging the area with 1.5 meters (5 feet) of temporary fill, is shown in the following table.

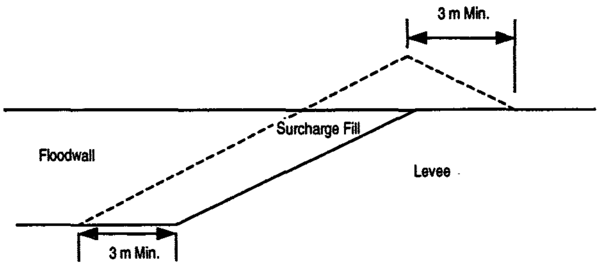
| Wall | Station | Cross Section Used in Analysis | Max. New Fill Height (m) | Estimated Time Rate Settlement (mm) | | | | Ultimate Settlement from New Fill (mm) | 90% Consolidation (mm) | Surcharge? ($\rho_{90\%}$ >25mm) | Time to 90% Consolidation (months) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 1 month | 3 months | 6 months | 1 year | | | | No Surcharge | w 1.5m Surcharge |
| FW6E | E20+407 | #31 | 0.7 | 25 | 37 | 46 | 56 | 75 | 68 | yes | 24+ | 1 |

As shown above, the time to achieve 90% consolidation is more than 24 months. If a surcharge height of 1.5 meters (5 feet) is placed for a period of about 1 month, 90% consolidation settlement could be achieved. This would result in post-surcharge settlement of less than 6 mm (¼ inch), assuming the walls are constructed after surcharge is removed. If surcharge is selected, we recommend that settlement plates/devices be installed prior to placement of the fill and settlement be monitored before, during and afterwards. These monitoring results should be submitted to and reviewed by URS.

Based on the estimated settlement, we estimate maximum differential settlement of the levee and cantilever L-type floodwall to be approximately 6 mm (¼ inch) in a distance of 25 meters (82 feet). The following sketches illustrate the lateral extent of the surcharge fill we recommend.



Typical Floodwall Cross-Section



Floodwall Profile A-A

If this estimated settlement is acceptable to the Structural Engineer and the District, then spread footing foundations would be appropriate. Otherwise, drilled piers are feasible, but should extend below the compressible foundation clay layers. Although the cantilever L-type floodwall will likely be used, recommendations for both foundation types are presented below.

## 7.4 DRIVING SIDE DESIGN EARTH PRESSURE

Consistent with the Corps requirements, a tension crack should be assumed to extend vertically down to the bottom of the shear key on the riverside. Therefore, there is no driving side earth pressure to be used for floodwall design. Only hydrostatic pressure need be considered (equal to 9.8 kN/$m^3$ or 62.4 pcf). This value is recommended, regardless of foundation type.

## 7.5 RESISTING SIDE DESIGN EARTH PRESSURE

### 7.5.1 At Rest Earth Pressure

The estimated moist unit weight of the clay soil adjacent to the floodwall is 20.4 kN/$m^3$ [130 pounds per cubic foot (pcf)]; and the corresponding submerged unit weight of the same clay soil is 10.7 kN/$m^3$ (68 pcf). Assuming that soil will not be excavated adjacent to the wall during its' lifetime, at-rest conditions occur on the resisting (landside) side; for this at-rest condition, a $K_o$ value of 0.66 with an equivalent fluid weight of 13.5 kN/$m^3$ (86 pcf) acting against the retaining walls is recommended. This assumes groundwater is temporarily at the bottom of the footing at the landslide edge. When groundwater rises to ground surface above the landside footing edge, an equivalent fluid weight of 16.8 kN/$m^3$ (107 pcf) is recommended against the retaining wall; this is for a combination of at-rest soil and water pressure. The wall resistance to sliding should be evaluated for both loading cases which represent different stages of seepage. The upper 300 mm (1 foot) of embedment should be neglected for design.

We understand floodwalls will be designed with a shear key. Where there is a shear key or the bottom of the slab is inclined, the shear resistance through the clay subgrade should be estimated using (1) a cohesion of 84 kPa (1,750 psf) (undrained condition); or (2) a friction angle of 24 degrees and an effective cohesion of 14 kPa (290 psf) (drained condition), whichever is less.

In the event that a spread footing (horizontal base) with no shear key is selected, an ultimate coefficient of friction of 0.325 (tangent 18 degrees for clay to concrete friction angle) should be used between the base of the retaining wall and underlying soil, with the total resistance not to exceed 36 kPa (750 psf). This assumes (1) that the retaining walls are cast neat against undisturbed engineered fill, (2) the bottom of slab is horizontal and (3) there is no shear key.

### 7.5.2 Passive Pressure

In accordance with the Corps manual for certain loading conditions, resisting-side earth pressure may be increased, but not to exceed one-half the (ultimate) passive pressure using unfactored shear strengths. Again, assuming that soil will not be excavated next to the wall during its' lifetime, an ultimate passive resistance of the soil should be estimated using a $K_p$ value of 3.2 with horizontal backfill and an equivalent fluid weight of 65.3 kN/$m^3$ (416 pcf); for a backfill sloped down at 2:1 we recommend a $K_p$ value of 1.7 with an equivalent fluid weight 34.7 kN/$m^3$ (221 pcf). This assumes groundwater is temporarily at the bottom of the footing at the landside edge. When groundwater rises to ground surface above the landside edge, an equivalent fluid weight of 34.2 kN/$m^3$ (218 pcf) for a horizontal backfill ($K_p = 3.2$) and 17.1 kN/$m^3$ (109 pcf) for a backfill sloped down at 2:1 is recommended against the retaining wall; this is for an ultimate passive resistance of soil only. Water pressure should be added to this value. The wall resistance to sliding should be evaluated for both loading cases which represent different stages of seepage. The upper 300 mm (1 foot) of embedment should be neglected for design.

## 7.6 BEARING CAPACITY

The ultimate bearing capacity for spread footings, when supported on new or existing engineered fill material consisting of stiff to very stiff clay soils, is 287 kPa [6,000 pounds per square foot (psf)]. In accordance with Corps EM 1110-2-2502, paragraph 4-4, page 4-1, the bearing capacity for loading Cases I1, I2, I3, and I4 should be equal to the ultimate bearing capacity divided by the appropriate factor of safety presented in Table 4-2 of that reference. The corresponding factors of safety for loading Cases I1, I2 and I4 are 3.0, 2.0 and 2.0, respectively. For Case I3, we recommend using a factor of safety of 1.5 which is consistent with Corp's requirements; Table 4-2 requires a minimum value greater than 1.0.

It is recommended that the bottom of footing be placed a minimum of 0.91 meters (3 feet) below lowest adjacent finished grade.

## 7.7 FOUNDATION UPLIFT

A spread footing foundation will be temporarily subject to seepage uplift during the 100-year flood event. We recommend the water pressure on the base be estimated by the line-of-creep method presented in Paragraph 3-19 of the Corps Manual EM 1110-2-2502. This method is based on estimates of the total heads at the ends of the wall base and then assuming that the total head varies linearly along the shortest possible seepage path along the wall base. As a tension crack is assumed extending the front toe of the wall, the total head will be equal to the water head at the design flood level; then, the uplift pressure is calculated by subtracting elevation head from the total head and multiplying the resulting pressure head by the unit weight of water. The back end of the wall is assumed to have no uplift pressure; then, the uplift pressure along the base will be linearly interpolated.

## 7.8 DRILLED PIERS

We understand there are no published Corps guidelines or manuals for design of drilled piers. Therefore, in the event drilled piers are selected by the Design Team using the Caltrans method (Memo to Designers 22-1, dated July 2001), we recommend the geotechnical design parameters in the following table.

| Geotechnical Parameter | Recommended Value |
|---|---|
| Design Unit Skin Friction for Piles | 24 kPa (500 psf) |
| Isolation Factor | 1.0 |
| Ka (active coefficient) | 0.36 |
| *Kp (passive coefficient) 2H:1V Slope Downward* | *1.7* |
| *Kp (passive coefficient) Horizontal* | *3.2* |

For pile embedment length less than 12 times the diameter, it can be assumed that the pile is relatively stiff and the depth of embedment can be determined by structural analysis consistent with the procedure outlined in the U.S.S. Steel Piling Design Manual (July 1975).

## 7.9 LATERAL LOAD CAPACITY OF DRILLED PIERS

The drilled pier foundation alternative will be capable of resisting lateral loads. The magnitude of the lateral load resistance is dependent on many factors, including the pile size embedment length, condition of pile cap fixity, the physical properties of the surrounding materials, and the yield moment capacity of the pile. To estimate the lateral load capacity of an individual drilled pier, we recommend using methods that are based on the p-y analysis. A commercially available computer program such as LPILE 4.0 (Reese, et al, October 2000) is a widely acceptable tool for this type of analysis. We understand that CH2MHILL is planning to use this program for the analysis of lateral load capacity of the drilled pier foundation alternative. The following table summarizes our recommended geotechnical input parameters for the LPILE program:

| Floodwall | Cross Section Number | Layer | LPILE Layer Type | Effective Unit Weight (kN/m³) | Soil Modulus Parameter, k (kN/m³) | Soil Shear Strength (kPa) | Strain at 50% Maximum Stress | Friction Angle (degrees) |
|---|---|---|---|---|---|---|---|---|
| **FW1E** | 32 | 1 | Stiff Clay with Free Water | 10.0 | 135,700 | 72 | 0.007 | - |
| | | 2 | Sand | 11.4 | 16,300 | - | - | 33 |
| | | 3 | Stiff Clay with Free Water | 10.5 | 27,100 | 48 | 0.01 | - |
| | | 4 | Sand | 11.4 | 5,400 | - | - | 33 |
| | | 5 | Stiff Clay with Free Water | 6.8 | 8,100 | 24 | 0.02 | - |
| **FW6E** | 31 | 1 | Stiff Clay with Free Water | 9.0 | 135,700 | 72 | 0.007 | - |
| | | 2 | Stiff Clay with Free Water | 10.0 | 27,100 | 36 | 0.01 | - |
| | | 3 | Stiff Clay with Free Water | 10.6 | 27,100 | 43 | 0.01 | - |

We recommend a group reduction factor of 0.7 for lateral pile capacity for pile groups with two or more rows of piles. These recommended group reduction factors are based on the assumption that the center-to-center spacing will be at least 3 times the pile diameter.

## 7.10 CONSTRUCTION CONSIDERATIONS FOR DRILLED PIERS

The conditions encountered in the borings consist primarily of layers of lean and fat clay with occasional lenses and layers of sand or gravel. Groundwater was encountered at about Elevation –1 to –2 meters, which corresponds to the approximate existing channel bottom. The August 2001 drawings also show a mean daily high tide at about Elevation 2.0 meters in this area. Therefore, during construction we would expect groundwater as high as Elevation 2.0 or higher depending on rainfall. While the cohesive deposits will help to keep the hole open during construction, some sloughing and caving can be expected if cohesionless deposits are encountered. Therefore, it is recommended that the Contractor have casing on site, should it be needed to minimize the risk of caving in either clay or sand soils. If casing is used, the casing should be withdrawn from the hole slowly as the concrete is being placed; a minimum head of concrete of 0.9 meter (3.0 feet) should be maintained in the casing at all times. All CIDH piles should be installed under the direct observation of a representative of the Geotechnical Engineer.

Based on the July 1, 2002 Design Submittal, it is anticipated floodwall FW6E will be constructed near an existing Mechanically Stabilized Embankment (MSE) wall. According to sheet S-23 of those submittal drawings, it appears that drilled piers associated with the construction of the floodwall will not encounter backfill or reinforcement straps from the existing MSE wall. However, detailed plans of the existing

MSE wall showing the as-built wall and strap configuration were not available for our review. Furthermore, we recommend the as-built drawings for the existing MSE wall be provided in the bid documents for the Contractor's information. Nevertheless, the Contractor should exercise caution during construction of the drilled piers so existing MSE wall straps and backfill are not disturbed.

## 7.11 CORROSION INVESTIGATION

An assessment of the potential for corrosion of various buried foundations and structures was performed by Villalobos & Associates. The results of their investigation are presented in Appendix D.

In summary, the field resistivity tests and laboratory testing indicate the following:

- The water soluble chloride level was 790 ppm; this level is considered to be in the corrosive category.
- The water soluble sulfate level was 335 ppm; this level is considered to be moderately corrosive.
- The pH of the tested boring sample was 7.2; this level is considered to be a neutral value.
- The level of bicarbonate ions was 918 ppm. High concentrations of bicarbonates tend to decrease soil resistivities. Bicarbonates are not aggressive to buried steel or concrete, but the lower resistivity environments can promote corrosion activity.
- The saturated (minimum) soil resistivity was 540 ohm-cm. The soil is considered to be corrosive at this location.

Based on the soil corrosivity data and review of the project requirements, corrosion monitoring and control are recommended to minimize corrosion of metallic and reinforced structures. These recommendations are in accordance with Caltrans Bridge Design Specifications, Section 8.22. The quality of materials and workmanship should be inspected during construction by a corrosion engineer.

### *Prestressed Concrete Piles*

Prestressed concrete piles should be constructed of durable concrete such as described in ACI Standards 201.2R and 222R. These recommendations include but are not limited to the following:

- The water/cement ratio should not exceed 0.40.
- A concrete cover of a minimum of 51 mm (2 inches) should be applied over all steel reinforcement.
- Mineral admixtures conforming to ASTM Designation C1240 and/or ASTM Designation C618 Type F and/or N should be used.
- Salt-free sand and potable water should be used.

- The concrete should be allowed to cure according to the manufacturer's recommendations.

### *Buried Metallic Structures*

Based on the low soil resistivity it is recommended that buried metallic piping systems, such as steel and ductile iron, be coated with a bonded dielectric or mortar coating. All non-welded joints should be bonded to provide electrical continuity. Cathodic protection should be installed to provide corrosion protection. Buried metallic piping should be electrically isolated from other structures, or where a change in piping or coating material occurs.

### *Buried Reinforced Concrete Structures*

Buried reinforced concrete structures should be constructed of durable concrete such as described in ACI Standards 201.2R and 222R. These recommendations include but are not limited to the following:

- The water/cement ratio should not exceed 0.40.
- A concrete cover of a minimum of 76 mm (3 inches) should be applied over all steel reinforcement.
- Salt-free sand and potable water should be used.
- The concrete should be allowed to cure according to the manufacturer's recommendations.

# SECTION 8

Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to the cyclic loading associated with earthquake ground shaking. Based on the latest ground motions anticipated for this site (Section 2 "Site Geology and Seismicity"), we have updated the evaluation of liquefaction potential performed previously for our April 2000 study. The current evaluation was based on semi-empirical methods considering the following three fault systems: 1) a moment magnitude 8.0 earthquake on the San Andreas fault, 2) a moment magnitude 7.1 on the Hayward fault and 3) a moment magnitude 6.5 on the Hayward Southeast Extension fault. The basis of these earthquake magnitudes and site accelerations is described in Section 2, "Site Geology and Seismicity."

The clay layers at the site are not susceptible to liquefaction. Therefore, the potential for liquefaction of various sand and gravel layers at the site was evaluated. On the basis of our analysis, it appears that there is some potential for liquefaction to occur at the locations shown on Figure 2, Location of Potentially Liquefiable Layers. The potentially liquefiable layers appear to be concentrated in the region of the levee alignment near the East Levee near the Alviso Marina, the UPRR tracks, and also the West Levee Overflow Weir. These layers range in thickness from 0.9 to 4.3 meters (3 to 14 feet). Should liquefaction occur, settlement could occur at these locations. Settlement of the ground surface is estimated to be on the order of 25 to 110 mm (1 to 4½ inches) at these locations. Estimates of these potential settlements were made using the procedures outlined by Tokimatsu and Seed, (1984).

Based on explorations conducted, most layers of potentially liquefiable soil were encountered below depths of at least 3 meters (10 feet) below ground surface at landside toe. These deposits are overlain by layers of non-liquefiable cohesive soils. Empirical research by Ishihara (1985) indicates that for level ground, liquefaction is not likely to result in visible damage at the ground surface as long as the cover of the liquefiable soil is equal to at least 3 meters (10 feet). Based on Ishihara's research and our local experience, we expect ground surface rupture is unlikely.

As discussed in our April 2000 report, we conclude that lateral spreading is unlikely at the liquefaction prone locations in this reach.

# SECTION 9

## 9.1 GENERAL

All site preparation and earthwork should be done under the observation of a representative of the Geotechnical Engineer and in accordance with the recommendations presented herein. In general, grading/earthwork will include raising crown and side slopes of existing levees, constructing a new overflow weir, excavating and backfilling for new floodwalls, constructing depressed maintenance roads and ramps and removal of sediment in the channel.

## 9.2 CLEARING AND DEMOLITION

All concrete flatwork, existing pavements, aggregate base, armoring aggregate, loose stone, drainage lines, underground utilities, trees, brush, vegetation, fencing and abandoned structures designated for removal should be demolished and removed from the site. Abandoned utilities encountered during grading should be removed in their entirety.

## 9.3 GRUBBING

Grubbing consists of the removal, within the levee foundations, floodwalls, maintenance road and ramp areas, of all stumps, roots, buried logs, old piling, old paving, drains, and other objectionable matter. Roots or other intrusions over 38 mm (1½ inches) in diameter within the levee foundations, floodwalls and maintenance road areas should be removed to a depth of 0.91 m (3 feet) below natural ground surface. Shallow tile drains sometimes found in agricultural areas should be removed from the levee foundation areas. Materials resulting from grubbing operations should be removed from the site. The sides of all holes and depressions caused by grubbing operations should be flattened before backfilling. Backfill consisting of material similar to adjoining soils, should be placed in layers up to the final foundation grade.

## 9.4 STRIPPING

After foundation clearing and grubbing operations are complete, stripping should be commenced. The purpose of stripping is to remove low growing vegetation and organic topsoil, including but not limited to the side slopes and crown of existing levees, the landside and waterside toe of existing levees, and the footprint of new levees and maintenance roads and ramps. The depth of stripping should be determined by local conditions and is expected normally to vary from 152 to 305 mm (6 to 12 inches). The final depth of stripping should be determined by a representative of the Geotechnical Engineer during site preparation. Materials resulting from stripping operations should be removed from the site. Organic topsoil that does not contain debris may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect. The stripped materials should not be re-used as engineered fill or blended with other materials.

## 9.5 REMOVAL OF SEDIMENT IN CHANNEL

The channel sediment areas should be excavated as shown on the drawings to bring those areas to their finish channel elevations. All sediment materials should be removed from the site and should not be re-used for construction of levees, maintenance roads, ramps and backfill for floodwalls. Organic topsoil that does not contain debris or contaminants may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect.

## 9.6 EXCAVATION

After the areas to receive fill have been properly cleared, grubbed and stripped, it is recommended that existing loose fills or soft deposits be removed to the underlying native soils and replaced with engineered fill. Floodwall areas should be excavated as required to bring those areas to their finish subgrade elevations. All loose soil should be removed from the exposed floodwall subgrade prior to wall construction. It should be noted that the excavation for floodwall FW6E subgrade may encounter horizontal reinforcement from the existing MSE wall. Should the horizontal reinforcement be encountered during subgrade excavation, the Structural Engineer should review the effect of disturbing or removing this reinforcement.

## 9.7 SUBGRADE PREPARATION

After demolition, stripping and subgrade excavations are complete, the exposed surface to be filled should further be prepared by scarifying the top 152 mm (6 inches) of the surface, moisture conditioning the material to near the optimum content, and recompacting to a minimum relative compaction of 90 percent of its maximum dry density in accordance with ASTM Test Designation D 1557. This procedure is intended to provide a tight bond between the foundation and fill, and to eliminate a plane of weakness at the interface. The foundation surface should be kept drained and not scarified until just prior to fill placement in order to minimize the risk of saturation from rainfall. The fill soils exposed in footing excavations for floodwalls are not expected to require scarification. However, the final need for scarification should be determined at the time of excavation.

At some locations, such as at toe of existing levees or depressed maintenance roads, the subgrade surfaces will be positioned near or below the free groundwater level or in areas which have been subjected to standing water. The exposed materials are expected to consist of low plasticity clays, silts and fine sands.

Control of the water in most shallow excavations in these materials probably could be accomplished by sloping them to drain and pumping from sumps within the excavated areas. After dewatering, some of the subgrade surfaces might be stable enough to support construction equipment; whereas, other areas might develop pumping or rutting. Please refer to Section 9.10 “Construction Dewatering”, for further discussion regarding this subject.

In order to minimize the potential for pumping subgrade conditions, it is recommended that the first lift of the fill be “track-walked” in place to a thickness of about 0.3 meter (1 foot) using a bulldozer or other track-mounted equipment. The initial lift should be dry of the laboratory optimum and compacted to a minimum relative compaction of 90 percent. Rubber tired equipment, such as scrapers or bottom dump trucks, should not be permitted to traverse the area. Pumping or rutting will be further exacerbated by repeated passes of rubber tired equipment. This could occur wherever a silt or clay subgrade is encountered in the existing levee fill or native soil.

In the event that deep seated pumping develops, it is recommended that the unstable subgrade areas be "bridged" using a combination of stabilization fabric (Mirafi 600X or equivalent) covered by a layer of granular, bridging material. The bridging material should consist of "bank run" or "crusher run" material conforming to the gradation discussed in Section 9.8.3.2 "Bridging Material." Since bridging material is highly permeable, seepage should be expected within this layer. Therefore, bridging material should only be used in areas where seepage is acceptable.

After the reinforcing fabric has been placed on the exposed subgrade surface, the bridging material should be track-walked into place over the fabric. It is estimated that a 0.46 to 0.61 meter (18 to 24 inch) thick layer of bridging material will be needed. Rubber tired equipment should not be permitted to traverse pumping areas until placement of the bridging material and stabilization fabric have been completed. Then additional fill can be placed and compacted to provide desired finished grades.

In areas where the silt or clay subgrade pumps or ruts (in levee fill or native soil), lime treatment is an alternative treatment to achieve stabilization. Since lime stabilization results in a relatively impervious soil, it can be used anywhere in the levee cross sections. Where subgrade areas are silty sand or clayey sand, they may require a combination of quicklime and flyash stabilization. The Contractor should retain his own testing laboratory to perform tests to determine the appropriate lime and/or flyash content; this should include unconfined compressive strength tests. It has been our experience on other local projects that 4 to 5 percent quicklime by weight will generally achieve the desired stabilization results. All lime/flyash stabilized subgrade should be prepared in accordance with Section 24 "Lime Stabilization" of the July 1992 Caltrans Standard Specifications. The Contractor should retain a Specialty Subcontractor experienced with lime stabilization. Furthermore, the Contractor should include this work in his bid price.

## 9.8 FILL MATERIALS

### 9.8.1 Approval

Soil materials, whether from sources on or off site, should be approved by a representative of the Geotechnical Engineer for the intended use and specifically for a required location or purpose. It is expected that most materials will be imported.

### 9.8.2 Onsite Materials

#### *9.8.2.1 Channel Sediment*

Where it is planned to remove portions of the channel (sediment), these materials should not be re-used as engineered fill. Organic topsoil that does not contain debris or contaminants may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect. Otherwise, these materials should be removed from the site.

#### *9.8.2.2 Excavation for Floodwalls*

The existing levee is engineered fill composed primarily of stiff to hard compacted lean clay with occasional fat clay. Where is it is planned to remove portions of the existing levee for new floodwall foundations, the materials could be reused as new levee fill providing they are properly processed to achieve a moisture content near the optimum prior to placement. The drying of materials with excessive moisture content should be in accordance with the requirements in Section 9.9 "Fill Placement and Compaction". However, where excavations for floodwall foundations are made into existing clay fill, these materials may be re-used; whereas if these excavations extend deeper into organic soils or other objectionable material, these materials should not be re-used as engineered fill.

#### *9.8.2.3 Existing Armoring Aggregate*

The existing surficial layer of armoring aggregate (on the crown, i.e. crest), should be removed. If left-in-place, this horizontal layer of aggregate material could allow seepage through and onto the landside slope. This material could be re-used for surfacing new maintenance roads, provided it is not contaminated or mixed with fine-grained clay or silt soils.

#### *9.8.2.4 Excavation for West Levee Overflow Weir*

The existing overflow weir appears to consist of fill material composed of stiff to very stiff lean clay with miscellaneous debris, concrete, brick, and wood material. Where it is planned to remove portions of the existing weir for construction of the new weir, these materials should not be reused as engineered fill.

### 9.8.3 Import Materials

#### *9.8.3.1 Levee and Weir Fill*

It is recommended import materials used for levee and weir construction consist of a soil or soil-rock mixture free from organic matter or other deleterious substances. The material should contain at least 50 percent by weight of clayey soils (passing the U.S. Standard No. 200 sieve). The clayey soils should be of low to medium plasticity as classified by the Unified Soil Classification System having a plasticity index within the range of 10 to 20 and a liquid limit less than 40. The material should not contain rocks or lumps over 152 mm (6 inches) in greatest dimension and should not contain more than 15 percent greater than 64 mm (2½ inches). Sand and gravel size particles should comprise no more than 50 percent of the soil-rock mixture. It is anticipated that materials meeting these general requirements are available from local quarries or borrow pits located offsite.

Fill for all maintenance roads should also meet these requirements in order to minimize seepage through the adjacent levee.

### *9.8.3.2 Bridging Material*

Bridging material, for bridging soft subgrades for maintenance road areas, should consist of a reasonably well graded mixture of coarse angular gravel and cobbles. The following gradation limits are recommended as a general design guideline.

| Sieve Size | | Percentage Passing |
|---|---|---|
| (mm) | (inches) | |
| 152 | 6 | 100 |
| 51 | 2 | 0 – 50 |
| 19 | ¾ | 0 – 10 |
| Durability Index: 25 minimum | | |

### *9.8.3.3 Armoring Aggregate*

Armoring aggregate for placement on the crown of the levees, maintenance roads and ramps, should consist of clean, hard and durable gravel or crushed rock, conforming to the following gradation.

| Sieve Size | | Percentage Passing |
|---|---|---|
| (mm) | (inches) | |
| 76 | 3 | 100 |
| 38 | 1½ | 75-55 |
| 19 | ¾ | 60-40 |
| 13 | ½ | 45-30 |
| 10 | 3/8 | 35-20 |
| 6 | ¼ | 25-15 |
| No. 200 | No. 200 | 20-0 |

The rock for armoring aggregate should have a minimum specific gravity of 2.40 in accordance with California Test Method No. 206 and a minimum durability index of 35 in accordance with California Test Method No. 229.

## 9.9 FILL PLACEMENT AND COMPACTION

All levee fills should be spread in uniform horizontal lifts not exceeding 203 mm (8 inches) in uncompacted thickness. Before compaction begins, the fill should be brought to a moisture content that will permit proper compaction by either aerating the material if it is too wet, or spraying the material with water if it is too dry. The lean clay or occasional fat clay should be compacted in the range from optimum to 3 percent wet of laboratory optimum. The native granular materials should be compacted near the laboratory optimum. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of water content. To prevent drying of the subgrade soils, placement of fill should start immediately after surface preparation and should proceed in a continuous operation until the site is brought to grade. No fill should be placed during rain or when saturation will hinder proper compaction. The finished grade surface of the compacted fill under floodwall footings should be kept in a moist condition prior to the placement of concrete. Jetting or flooding of the fill should not be permitted.

Relative compaction is the ratio of the in place dry density of the constructed fill to a maximum dry density determined by ASTM Test Designation D 1557. All levee fills should be compacted to a minimum relative compaction of 90 percent, as determined by ASTM Test Designation D 1557. All pavement aggregate base course, subbase and subgrade materials, and any fill material which is within the upper 600 mm (2 feet) below finished pavement surface should be compacted to a minimum relative compaction of 95 percent. All fill material below the bottom of footings and all other structural elements should be compacted to a minimum relative compaction of 95 percent. All other fill materials should be compacted to a minimum relative compaction of 90 percent. A representative of the Geotechnical Engineer should be present to observe all grading operations during both preparation of the site and compaction of engineered fill.

The compaction and moisture content of the subgrade and each layer of levee placed should be tested by a representative of the Geotechnical Engineer in accordance with ASTM D 1556 "Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method" or ASTM D 2922 "Standard Test Method for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)", ASTM D 3017 "Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)" and ASTM D 2216 "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock."

Fills placed on slopes steeper than 6:1 (horizontal to vertical) should be benched into the adjacent soils to provide a bond at the interface. Loose soils on the surface of existing slopes should be excavated as new fill is placed. Benching is expected to be necessary where new fill will be placed on existing levee slopes in order to raise the existing levee. It is recommended that horizontal benches be constructed such that the maximum height of the vertical section of the bench does not exceed 305 mm (1 foot).

Where new fill is placed on the face of an existing levee, the fill should extend laterally a minimum of 0.61 meter (2 feet) beyond the finished lines during construction. After compaction, the excess fill should be removed to the final grade and lines.

## 9.10 CONSTRUCTION DEWATERING

Because the groundwater level is below the bottom of the anticipated depth of excavations for floodwalls, the excavation bottoms are expected to be relatively dry during the excavation phase. In the event that water becomes ponded during the rainy season, the ponded water could be removed by sump pit/pump procedures.

Depending on the season of the year, the water level could vary considerably in the Guadalupe River channel. It is conceivable that some riverside subgrade areas may be soft and wet. As discussed previously, those areas should be over-excavated and bridging material placed to a maximum depth of 0.61 meter (2 feet); alternatively, lime stabilization treatment could be used. The water encountered could be removed by sump pit/pump procedures.

# SECTION 10

## 10.1 GENERAL

Based on our site reconnaissance, as well as our subsurface investigation and geotechnical analyses, we believe the geotechnical condition of the existing East Levee between the Alviso Marina and UPRR bridge as described herein is generally very good. These opinions and our recommendations are discussed below in more detail in this section.

## 10.2 RESULTS OF LEVEE SLOPE STABILITY EVALUATION

In Section 4 "Levee Slope Stability Evaluation", we discussed (1) the cross sections selected for analysis, (2) method of analysis and (3) stability results. We identified two (2) levee sections as candidates for analysis. Based on our slope stability analyses of these 2 sections, we conclude that both of these sections had factors of safety that met or exceeded the minimum factor of safety required for four (4) Corps' design conditions. However, based on earthquake loading conditions, we anticipate permanent deformation of the levees to be in the range of 0 to 284 mm (0 to 11 inches).

As previously stated, it is unlikely that the design earthquake event will coincide with a period of high water. However, all levees should be inspected following a major seismic event since deformations due to earthquake shaking are anticipated. Any levee repairs and/or upgrades should be consistent with the District's levee safety program.

## 10.3 RESULTS OF SETTLEMENT ANALYSES

In Section 5 "Settlement Evaluation", we discussed (1) land subsidence, (2) inferred levee settlement, (3) predicted settlement, (4) future levee settlement and (5) West Levee Overflow Weir settlement. The future levee and weir settlement is based on consolidation of the foundation clay soil due to previous filling in 1963 and 1983, as well as filling due to proposed levee and weir raise. On that basis, we estimate the future maximum long term consolidation settlement to be on the order of 168 mm (6½ inches) for the East Levee and 676 mm (26½ inches) for the West Levee Overflow Weir (assuming existing fill is not removed and is compressible). Should removing and replacing the existing fill at the weir be considered, we estimate the future long term consolidation settlement to be on the order of 627 mm (24½ inches). The levee and weir should be overbuilt to accommodate this future settlement.

We estimate maximum differential settlement of the levee and floodwall (placed on spread footings) to be about 18 mm (¾ inch) in a distance of 10 m (33 feet). If this estimated settlement is acceptable to the Structural Engineer and the District, then spread footing foundations are recommended. The floodwall should be overbuilt to accommodate this future settlement.

If these settlements at floodwall locations are not acceptable to the Structural Engineer and the District, then drilled piers are recommended for the support of the floodwalls. Drilled piers should extend below the compressible foundation clay layers, however. In addition, a concrete cutoff wall should extend a minimum of 1.0 meter (3.3 feet) below lowest adjacent finished grade. Since the levee will settle relative to the floodwall, periodic grouting may be required to fill any voids between the wall foundation and levee fill soil.

## 10.4 RESULTS OF SEEPAGE ANALYSES

In Section 6 “Seepage Evaluation” we discussed (1) method of analysis and (2) cross sections and results of analysis. In general, the East Levee is constructed of well compacted lean clays with some localized fat clays. Furthermore, most of the native soils underlying the levee fills are cohesive. Since the cohesive soils are relatively impervious, it is our engineering opinion that generally (1) seepage through and underneath the levees during the design flood will be relatively small and (2) the potential for piping at the outboard toe will also be low. To confirm these opinions, we performed a seepage analysis on two cross sections.

For a 30.5-meter (100-foot) long section, the total flow quantity through the East Levee at each of the two cross sections is expected to be less than 1/10 gallon per minute (gpm). In our opinion these seepage quantities are very small. In particular, the seepage analysis we performed for Cross Section 32 assumes the maintenance road fill is low permeability material consisting of lean clay.

The above discussion is for a steady state condition such that seepage through the levee achieves equilibrium during the 100-year flood elevation. To simulate the change in river level during the flood event, we also performed a transient analysis based on the 100-year flood hydrograph. Based on this analysis, the highest phreatic surface is at 48 hours; this surface is lower than the steady state phreatic surface. By comparison, we conclude that the estimated seepage quantities probably will be less than the estimated values at the 2 cross sections.

We also recommend that prior to raising the crest of any existing levee, the surficial layer of armour aggregate be removed. If left-in-place, this horizontal layer of aggregate material could allow seepage through and onto the landside slope.

## 10.5 SITE PREPARATION AND GRADING

In Section 9 “Site Preparation and Earthwork” we discussed in detail the following earthwork aspects: (1) clearing and demolition, (2) grubbing, (3) stripping, (4) removal of sediment in channel, (5) excavation, (6) subgrade preparation, (7) fill materials, including onsite materials and import materials, (8) fill placement and compaction, and (9) construction dewatering. Please refer to that section for details.

## 10.6 LIQUEFACTION POTENTIAL

In Section 8, “Liquefaction Potential”, we presented our updated evaluation of liquefaction potential of the site and concluded that there are locations that have a relatively high potential of liquefaction during major earthquake shaking. These locations are presented graphically in Figure 2. However, it is our opinion that lateral spreading due to liquefaction is unlikely. The likely consequence that may impact levees will be liquefaction induced settlement; we estimate such settlement to be on the order of

25 to 110 mm (1 to 4½ inches). Therefore we recommend that the District inspect the levee after major earthquake events to assess the need for raising the settled crest of the levee and/or grouting underneath floodwalls if gaps result from the levee settlement. In any event, the inspection and mitigation should be consistent with the District's Levee Safety Technical Guidance Manual.

SECTION 11

The conclusions and recommendations contained in this report are based on exploratory borings drilled for this study, as well as our review of available subsurface information, our previous site reconnaissance, and our local experience and engineering judgment. Furthermore, the slope stability analysis has been limited to the two (2) critical cross sections selected.

Surveyed ground elevations of borings were not available. Elevations were estimated from nearby cross section geometry provided by the District and from spot elevations provided by CH2MHILL.

An investigation for subsurface environmental contamination was beyond the scope of our services.

If any variations or undesirable conditions are encountered during construction, URS should be contacted so that supplementary recommendations can be made. Also, if the construction plans are changed from those presently conceived, URS should review the changes and make modifications to the original recommendations presented in the report in order to meet the project needs.

Geotechnical issues may arise during construction which are not apparent at this time. URS should be retained during construction to review the soil conditions encountered and the construction procedures. All pile foundation installation and testing should be done under the direct observation of a URS representative.

The recommendations presented in this report were developed with the standard of care commonly used in this profession. No other warranties are included, either express or implied, as to the professional advice presented in this report.

# SECTION 12

Abrahamson, N.A. and Silva, W.J., 1997, Empirical response spectral attenuation relations for shallow crustal earthquakes, Seismological Research Letters, v. 68, p. 94-127.

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1993, Estimation of response spectra and peak accelerations from western North American earthquakes: An interim report, U.S. Geological Survey Open-File Report 93-509, 72 p.

Bureau of Reclamation, U.S. Department of the Interior, 1974, Earth Manual, Second Edition.

Campbell, K.W., 1997, Empirical near-source attenuation relationships for horizontal and vertical components of peak ground acceleration, peak ground velocity, and pseudo-absolute acceleration response spectra, Seismological Research Letters, v. 68, p. 154-179.

Cedergren H.R., 1967, Seepage, Drainage and Flow Notes, 489 p.

Department of the Army Corps of Engineers, 1970, Engineering and Design - Stability of Earth and Rock-Fill Dams, Manual EM-1110-2-1902.

Department of the Army Corps of Engineers, 1978, Design and Construction of Levees, Manual EM-1110-2-1913.

Department of the Army Corps of Engineers, September 30, 1986, Seepage Analysis and Control of Dams, Manual EM-1110-2-1901.

Department of the Navy Naval Facilities Engineering Command, 1982, Soil Mechanics Design Manual 7.1.

Duncan, J.M. and Buchignani, A.L., 1975, An engineering manual for slope stability studies, University of California, Berkeley, Department of Civil Engineering.

Geo-Slope International, Ltd., 2001 User's Guide, SLOPE/W for Slope Stability Analysis, Version 4.24.

Geo-Slope International, Ltd., 1998, User's Guide, SEEP/W for Seepage Analysis, Version 4.21.

Hitchcock, C.S., Kelson, K.I. and Thompson, S.C., 1994, Geomorphic investigations of deformation along the northeastern margin of the Santa Cruz Mountains, William Lettis & Associates, Inc., Final Technical Report, U.S. Geological Survey National Earthquake Hazards Reduction Program Award No. 1434-92-G-2220, 33 p.

Irwin, W.P., 1990, Geology and plate tectonic development, in The San Andreas Fault System, California, Wallace, R.E. (ed.) U.S. Geological Survey Professional Paper 1515, p. 61-80.

Jennings, C.W., 1994, Fault activity map of California and adjacent areas, California Division of Mines and Geology, California Geologic Data Map Series, Map No. 6, 1:750,000 scale.

Leonards, G.A., 1962, Foundation Engineering, McGraw-Hill Book Company.

Lienkaemper, J.J., Borchardt, G. and Lisowski, M., 1991, Historic creep rate and potential for seismic slip along the Hayward fault, California, Journal of Geophysical Research, v. 96B, p. 18261-18283.

Lienkaemper, J.J. and Galehouse, J.S., 1997, Revised long-term creep rates on the Hayward fault, Alameda and Contra Costa Counties, California, U.S. Geological Survey, Open-File Report 97-690, 18 p.

Poland, J.F., 1971, Land Subsidence in the Santa Clara Valley, Alameda, San Mateo, and Santa Clara Counties, California, U. S. Geological Survey, Map MF - 336.

Rogers, T .H and Williams, J.W., 1974, Potential Seismic Hazards in Santa Clara County, California, California Division of Mines and Geology, Special Report 107, 39p.

Sadigh, K., Chang, C.Y., Egan, J.A., Makdisi, F. and Youngs, R.R., 1997, Attenuation relationships for shallow crustal earthquakes based on California strong motion data, Seismological Research Letters, v. 68, p. 180-189.

Seed, R.B. and Harder, L.F., May, 1990, SPT-Based analysis of cyclic pore pressure generation and undrained residual strength, Volume 2, H. Bolton Seed Memorial Symposium Proceedings.

URS Greiner Woodward Clyde, 2000. "Geotechnical Investigation, Lower Guadalupe River Flood Control Project, Santa Clara County, California." Job No. 95-973017NB.00. Dated April 28, 2000.

Wallace, R.E.(ed.), 1990, The San Andreas Fault System, California, USGS, Professional Paper 1515, 283 p.

Working Group on California Earthquake Probabilities, 1990, Probabilities of large earthquakes in the San Francisco Bay region, California, U.S. Geological Survey, Circular 1053, 51 p.

Working Group on Northern California Earthquake Potential, 1996, Database of potential sources for earthquakes larger than magnitude 6 in northern California, U.S. Geological Survey, Open-File Report 96-705, 53 p.

TABLES

# TABLE 1
# Lower Guadalupe River Flood Control Project
# EXISTING BORING LOG INDEX LIST

| **Boring Index** | **Consultants** | **Consultants' Project Number** | **Project Date** | **Project Name** |
|---|---|---|---|---|
| AA-n | Woodward-Clyde | 973061NA | Sept 1997 | Preliminary Investigation, Fuel Storage and Dispensing Facility for American Airlines, San Jose |
| BSP-n | Woodward-Clyde | 4327 | Dec 1960 | Soil Investigation for the Proposed Bay Shell Plant Storage Tanks, San Jose |
| GJJ-n | Gribaldo,Jacobs, Jones & Associates | E584-M1 | Jan 1963 | Soil Investigation for Guadalupe River Project, Reach No. 1, Preliminary Study of the West Bank of Alviso Slough and Proposed SPRR Retaining Wall, Santa Clara County |
| GRI-n | John V. Lowney & Associates | 109-11<br>PA 9669 | Jan 1981 | Geotechnical Investigation, Guadalupe River Improvement, San Jose |
| GRIM-n | Woodward-Clyde | 15482V | Oct 1982 | Geotechnical Investigation, Guadalupe River Flood Control Improvements, Southern Pacific Railroad to the County Marina in Alviso |
| GRIV-n | Woodward-Clyde | 4246 | c. 1960 | Soil Investigation for the Proposed Guadalupe River Project, Reaches 1 and 2, Santa Clara County |
| GROCS-n | John V. Lowney & Associates | 109-12<br>PA 11516 | June 1983 | Geotechnical Investigation, Guadalupe River Outfall Control Structure, San Jose |
| GSB-n | Woodward-Clyde | 16338V | Oct 1984 | Geotechnical Investigation, Raising of Gold Street Bridge over the Guadalupe River in Alviso, San Jose |
| GSBOLD-n | Woodward-Clyde | 4246 | c. 1960 | Gold Street Bridge Investigation as part of the 1960 Guadalupe River Project |
| GTPP-n | Parikh Consultants, Inc. | 953009NE | c. 1996 | Golden Triangle Pipeline Project, South Bay Water Recycling Program |
| HHP-n | Woodward-Clyde | 91C0639R | Jan 1992 | "Hetch-Hetchy Pipelines," Relocation of San Francisco Water Department, Bay Division Pipeline Numbers 3 and 4 at Guadalupe River and Coyote Creek, San Jose and Santa Clara |
| HHPA-n | PSC Associates, Inc. | 88122.10 | July 1988 | Lick Mill Blvd. Extension, Santa Clara |
| OPS-n | Woodward-Clyde | 16784V | c. 1985 | Geotechnical Consultation, Oakmead Storm Pump Station Outfall Spillway |

# TABLE 1
# Lower Guadalupe River Flood Control Project
# EXISTING BORING LOG INDEX LIST

| Boring Index | Consultants | Consultants' Project Number | Project Date | Project Name |
|---|---|---|---|---|
| PRMAR-n | Soil Mechanics & Foundation Engineers | 1140 | Feb 1962 | Soil Engineering Report for the Proposed Marina, Alviso, Santa Clara County |
| PYH-n | Cooper & Clark | 33A | Jan 1960 | Preliminary Site Investigation, Proposed Yacht Harbor, Alviso |
| RBPS-n | Woodward-Clyde | 16171X | Feb 1987 | Geotechnical Investigation, Retention Basin and Pump Station, San Jose International Airport, San Jose |
| REAL1-n | Woodward-Clyde | 5135 | June 1962 | Soil Investigation, Guadalupe River Project, Realignment within Reach 1, Santa Clara |
| RHM-n | Woodward-Clyde | 92C0526R | Nov 1992 | Soil Investigation, Riparian Habitat Mitigation, Route 87 |
| RREMB-n | John V. Lowney & Associates | 109-1<br>PA 1233 | Oct 1969 | Soil Mechanics Investigation, SPRR Bridge and Embankment Raising over Guadalupe River, San Jose |
| SBYC-n | Woodward-Clyde | 15482-W-2000 | April 1983 | Geotechnical Study, South Bay Yacht Club Relocation, Alviso |
| SDB-n | Woodward-Clyde | 963010NA | c. 1997 | Geotechnical Investigation, Skyport Drive Bridge, San Jose |
| SIL-n | Cooper & Clark | 170-G | Oct 1982 | Soil Investigation, Landslide Areas, Guadalupe River, San Jose |
| SJIA-n | Woodward-Clyde | 16171V | Feb 1984 | Geotechnical Investigation, San Jose International Airport, San Jose |
| SPRR-n | Jo Crosby & Associates | 2706 | Jan 1976 | Soils Investigation for Guadalupe River, SPRR to Trimble Road, Project # 3015 |
| SPRRB-n | Woodward-Clyde | 4246 | Jan 1961 | Soil Investigation for the Proposed Southern Pacific Company Bridge, Alviso (in conjunction with soil investigation of Guadalupe River Project, Reaches 1 and 2) |
| TDB-n | Woodward-Clyde | 16266V | May 1984 | Geotechnical Study, Tasman Drive Bridge at Guadalupe River, Santa Clara/San Jose |

Note:

1. The value "n" in the boring index column represents the Boring Number used in the corresponding project, such as 1 and B1.

# TABLE 2a

## ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 1

| Fault | Maximum Magnitude ($M_w$) | Sense of Faulting[1] | Distances | | | Peak Ground Acceleration (g) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Horizontal Distance[2] (km) | Seismogenic Distance[3] (km) | Rupture Distance[4] (km) | Abrahamson & Silva (1997) | Campbell (1997) | Sadigh *et al.* (1997) | Boore *et al.* (1997) | Average |
| Monte Vista - Shannon | 6.8 | R | 13.8 | 15.9 | 13.8 | 0.45 | 0.46 | 0.48 | 0.45 | **0.46** |
| Hayward - SE Extension | 6.5 | OR | 9.4 | 11.6 | 9.4 | 0.57 | 0.56 | 0.58 | 0.50 | **0.55** |
| Hayward | 7.1 | SS | 10.6 | 10.8 | 10.6 | 0.46 | 0.58 | 0.48 | 0.56 | **0.52** |
| Calaveras | 6.7 | SS | 13.8 | 14.0 | 13.8 | 0.36 | 0.42 | 0.37 | 0.38 | **0.38** |
| San Andreas - Peninsula Segment | 7.2 | SS | 20 | 20.1 | 20 | 0.30 | 0.40 | 0.33 | 0.38 | **0.36** |
| San Andreas - Santa Cruz Mountains Segment | 7.2 | SS | 20 | 20.1 | 20 | 0.30 | 0.40 | 0.33 | 0.38 | **0.36** |
| San Andreas - 1906 Rupture | 8 | SS | 20 | 20.1 | 20 | 0.38 | 0.58 | 0.45 | 0.58 | **0.50** |
| Sargent | 6.8 | OR | 17.8 | 19.9 | 17.8 | 0.37 | 0.36 | 0.40 | 0.38 | **0.38** |

---

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] $M_w$ and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] $M_w$ and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] $M_w$ and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

# TABLE 2b

## ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 2

| Fault | Maximum Magnitude ($M_w$) | Sense of Faulting[1] | Distances | | | Peak Ground Acceleration (g) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Horizontal Distance[2] (km) | Seismogenic Distance[3] (km) | Rupture Distance[4] (km) | Abrahamson & Silva (1997) | Campbell (1997) | Sadigh *et al.* (1997) | Boore *et al.* (1997) | Average |
| Monte Vista - Shannon | 6.8 | R | 14.4 | 16.4 | 14.4 | 0.43 | 0.45 | 0.47 | 0.43 | **0.45** |
| Hayward - SE Extension | 6.5 | OR | 9.4 | 11.6 | 9.4 | 0.57 | 0.56 | 0.58 | 0.50 | **0.55** |
| Hayward | 7.1 | SS | 11.9 | 12.1 | 11.9 | 0.42 | 0.55 | 0.45 | 0.52 | **0.48** |
| Calaveras | 6.7 | SS | 13.1 | 13.3 | 13.1 | 0.38 | 0.44 | 0.38 | 0.39 | **0.40** |
| San Andreas - Peninsula Segment | 7.2 | SS | 20 | 20.1 | 20 | 0.30 | 0.40 | 0.33 | 0.38 | **0.36** |
| San Andreas - Santa Cruz Mountains Segment | 7.2 | SS | 20 | 20.1 | 20 | 0.30 | 0.40 | 0.33 | 0.38 | **0.36** |
| San Andreas - 1906 Rupture | 8 | SS | 20 | 20.1 | 20 | 0.38 | 0.58 | 0.45 | 0.58 | **0.50** |
| Sargent | 6.8 | OR | 17.5 | 19.6 | 17.5 | 0.37 | 0.37 | 0.41 | 0.38 | **0.38** |

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] $M_w$ and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] $M_w$ and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] $M_w$ and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

# TABLE 2c

## ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 3

| | | | Distances | | | Peak Ground Acceleration (g) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Fault | Maximum Magnitude ($M_w$) | Sense of Faulting[1] | Horizontal Distance[2] (km) | Seismogenic Distance[3] (km) | Rupture Distance[4] (km) | Abrahamson & Silva (1997) | Campbell (1997) | Sadigh *et al.* (1997) | Boore *et al.* (1997) | Average |
| Monte Vista - Shannon | 6.8 | R | 13.8 | 15.9 | 13.8 | 0.45 | 0.46 | 0.48 | 0.45 | **0.46** |
| Hayward - SE Extension | 6.5 | OR | 9.4 | 11.6 | 9.4 | 0.57 | 0.56 | 0.58 | 0.50 | **0.55** |
| Hayward | 7.1 | SS | 13.8 | 13.9 | 13.8 | 0.38 | 0.50 | 0.41 | 0.47 | **0.44** |
| Calaveras | 6.7 | SS | 13.8 | 13.9 | 13.8 | 0.36 | 0.42 | 0.37 | 0.38 | **0.38** |
| San Andreas - Peninsula Segment | 7.2 | SS | 19.5 | 19.6 | 19.5 | 0.31 | 0.41 | 0.34 | 0.39 | **0.36** |
| San Andreas - Santa Cruz Mountains Segment | 7.2 | SS | 19.5 | 19.6 | 19.5 | 0.31 | 0.41 | 0.34 | 0.39 | **0.36** |
| San Andreas - 1906 Rupture | 8 | SS | 19.5 | 19.6 | 19.5 | 0.39 | 0.59 | 0.46 | 0.59 | **0.51** |
| Sargent | 6.8 | OR | 16.9 | 19.0 | 16.9 | 0.38 | 0.38 | 0.42 | 0.39 | **0.39** |

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] $M_w$ and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] $M_w$ and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] $M_w$ and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

# TABLE 2d

## ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 4

| Fault | Maximum Magnitude ($M_w$) | Sense of Faulting[1] | Distances |  |  | Peak Ground Acceleration (g) |  |  |  |  |
|---|---|---|---|---|---|---|---|---|---|---|
|  |  |  | Horizontal Distance[2] (km) | Seismogenic Distance[3] (km) | Rupture Distance[4] (km) | Abrahamson & Silva (1997) | Campbell (1997) | Sadigh *et al.* (1997) | Boore *et al.* (1997) | Average |
| Monte Vista - Shannon | 6.8 | R | 12.5 | 14.6 | 12.5 | 0.48 | 0.50 | 0.52 | 0.48 | **0.49** |
| Hayward - SE Extension | 6.5 | OR | 9.4 | 11.6 | 9.4 | 0.57 | 0.56 | 0.58 | 0.50 | **0.55** |
| Hayward | 7.1 | SS | 16.3 | 16.4 | 16.3 | 0.34 | 0.45 | 0.37 | 0.42 | **0.40** |
| Calaveras | 6.7 | SS | 13.1 | 13.3 | 13.1 | 0.38 | 0.44 | 0.38 | 0.39 | **0.40** |
| San Andreas - Peninsula Segment | 7.2 | SS | 18.8 | 18.9 | 18.8 | 0.32 | 0.42 | 0.35 | 0.40 | **0.37** |
| San Andreas - Santa Cruz Mountains Segment | 7.2 | SS | 18.8 | 18.9 | 18.8 | 0.31 | 0.41 | 0.34 | 0.39 | **0.36** |
| San Andreas - 1906 Rupture | 8 | SS | 18.8 | 18.9 | 18.8 | 0.31 | 0.41 | 0.34 | 0.39 | **0.36** |
| Sargent | 6.8 | OR | 16.3 | 18.4 | 16.3 | 0.39 | 0.40 | 0.43 | 0.40 | **0.40** |

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] $M_w$ and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] $M_w$ and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] $M_w$ and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

# TABLE 3
## Lower Guadalupe River Flood Control Project
## SUBSURFACE CHARACTERIZATION
## LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT

| Levee | Levee Section Number | Station Limits | | Comments |
|---|---|---|---|---|
| | | From | To | |
| West | W1 | 7+000 | 7+450 | |
| | W2 | 7+450 | 8+200 | Common with E2 |
| | W3 | 8+200 | 8+500 | |
| | W4 | 8+500 | 9+230 | |
| | W5 | 9+230 | 10+100 | |
| | W6 | 10+100 | 11+730 | |
| | W7 | 11+730 | 13+000 | |
| | W8 | 13+000 | 14+040 | |
| | W9 | 14+040 | 14+570 | Common with E9 |
| | W10 | 14+570 | 15+450 | Common with E10 |
| | W11 | 15+450 | 15+900 | |
| | W12 | 15+900 | 16+770 | |
| | W13 | 16+770 | 17+550 | |
| East | E1 | 6+200 | 6+640 | |
| | E2 | 6+640 | 8+050 | Common with W2 |
| | E3 | 8+050 | 9+550 | |
| | E4 | 9+550 | 10+140 | |
| | E5 | 10+140 | 11+100 | |
| | E6 | 11+100 | 11+440 | |
| | E7 | 11+440 | 12+450 | |
| | E8 | 12+450 | 13+650 | |
| | E9 | 13+650 | 14+850 | Common with W9 |
| | E10 | 14+850 | 15+56 | Common with W10 |

## Table 4
## Lower Guadalupe River Flood Control Project
## PROPOSED 2 CROSS SECTIONS OF LEVEES CONSIDERED TO BE MOST CRITICAL

| Cross Section Number | Relevant Levee Section Number | Approximate Station | Section Location | Relevant Boring(s) | Comments |
|---|---|---|---|---|---|
| 31 | E1 | 6+300 | East Levee | EB-28, EB-01, GJJ-3 | levee raise, fat clay |
| 32 | E2 | 6+780 | East Levee | EB-27, CB-14, CB-15 | levee raise, fat clay, maintenance road |

# TABLE 5
# Lower Guadalupe River Flood Control Project
# SUMMARY OF EFFECTIVE SHEAR STRENGTH PARAMETERS BASED ON ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TESTS

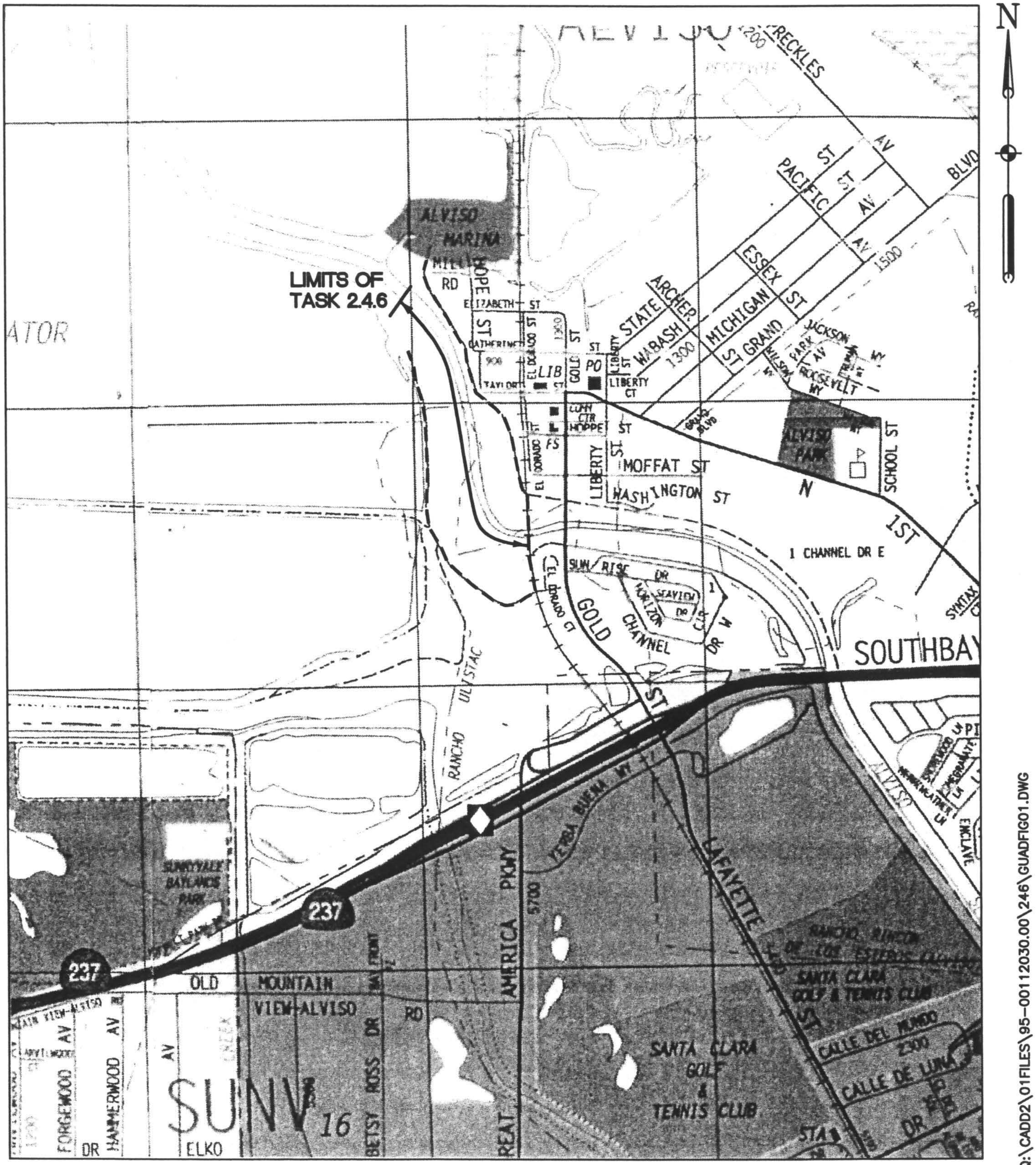
| SOIL TYPE | DRY DENSITY RANGE (pcf) | ESTIMATED EFFECTIVE STRENGTH PARAMETERS | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | $\phi'$ (degrees) | | | | c' (psf) | | | |
| | | Peak | 10% $\varepsilon$ | 20% $\varepsilon$ | Design | Peak | 10% $\varepsilon$ | 20% $\varepsilon$ | Design |
| Lean CLAY FILL | Less than 110 | 23.7 | 22 | 16.3 | 24 | 456 | 466 | 574 | 450 |
| Lean CLAY FILL | Greater than 110 | 32.2 | 33.8 | 30.1 | 33 | 0 | 0 | 0 | 0 |
| Fat CLAY | Less than 80 | 19.8 | 12.2 | NA | 20 | 113 | 214 | NA | 110 |
| Lean CLAY | From 80 to 100 | 30.4 | 29 | 26.7 | 30 | 35.1 | 182.6 | 186.5 | 35 |
| Lean CLAY | From 100 to 110 | 29.4 | 30 | 25.3 | 30 | 153.4 | 144.8 | 424.2 | 150 |

Note: Definitions for attached figures are

$q = \frac{1}{2} (\sigma_1' - \sigma_3')$

$p = \frac{1}{2} (\sigma_1' + \sigma_3')$

# FIGURES



NOT TO SCALE

Q:\CADD2\01FILES\95-00112030.00\246\GUADFIG01.DWG

| | | | |
|---|---|---|---|
| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | SITE MAP | Figure<br>1 |
| 5/31/02 | | | |
| URS | | | |

ALVISO
SECTION E1
SECTION E2
SECTION E3
SECTION E4
SECTION E5
SECTION E6
SECTION E7
SECTION E8
SECTION E9
SECTION E10
SECTION W1
SECTION W2
SECTION W3
SECTION W4
SECTION W5
SECTION W6
SECTION W7
SECTION W8
SECTION W9
SECTION W10
SECTION W11
SECTION W12
SECTION W13
APPROXIMATE CHANNEL LIMITS OF OLD MEANDERS (1960, 1963)
APPROXIMATE CHANNEL LIMITS OF OLD MEANDERS (1873)
APPROXIMATE CENTER LINE OF EAST LEVEE
APPROXIMATE CENTERLINE OF WEST LEVEE
CHANNEL ALIGNMENT
VTA MITIGATION LEVEE
SETBACK LEVEE
UPRR
GOLD STREET
HWY 237
TASMAN DR.
MONTAGUE EXPY
TRIMBLE RD.
US 101
AIRPORT PARKWAY
SKYPORT DRIVE
N
SCALE IN METERS
200 0 200
LEGEND:
APPROXIMATE LOCATION OF EXPLORATORY BORING, 2001 AND 2002
APPROXIMATE LOCATION OF EXPLORATORY BORING, 1998
APPROXIMATE LOCATION OF CONE PENETRATION TEST, 1998
⑦ CROSS SECTION ANALYZED BY URS (2000)
(30) CROSS SECTION ANALYZED FOR THIS STUDY
LOCATION OF POTENTIALLY LIQUEFIABLE LAYER

SOURCE: CHANNEL ALIGNMENT IS BASED ON AUTOCAD DRAWING PROVIDED BY SCVWD.

Q:\CADD2\01FILES\95-00112030.00\PLAN\GUADPLAN02.DWG

| REV | DESCRIPTION OF REVISION | BY | DATE |
|---|---|---|---|
| △ | MOVED LOCATION OF SEVERAL BORINGS AND CPTS AND REVISED NUMBERS. | S.H. | 3/19/98 |
| △ | MOVED WC-4 AND WB-5 PER CULTURAL RESOURCE RECOMMENDATION. | S.H. | 4/9/98 |
| △ | ADDED CROSS SECTIONS 8 THROUGH 30 AND LEVEE SECTIONS. | M.L.L. | 9/18/01 |
| △ | ADDED BORING LOCATIONS (2001 STUDY) AND VTA MITIGATION LEVEE | J.W. | 11/05/01 |
| △ | ADDED NEW BORING LOCATIONS, SETBACK LEVEE, CROSS SECTIONS 31 THROUGH 56 | T.P. | 5/29/02 |
| △ | | | |

7/1/02

**URS**

55 South Market Street, Suite 1650
San Jose, Ca. 95113

| | |
|---|---|
| DESIGNED | T.P. |
| DRAWN | E.G. |
| CHECKED | S.H. |
| PEER REVIEWED | M.L. |
| PROJECT MANAGER | P.B. |
| DATE | 7/1/02 |

Lower Guadalupe River Flood Protection Project Santa Clara County, CA.

# SITE AND FIELD EXPLORATION LOCATION PLAN

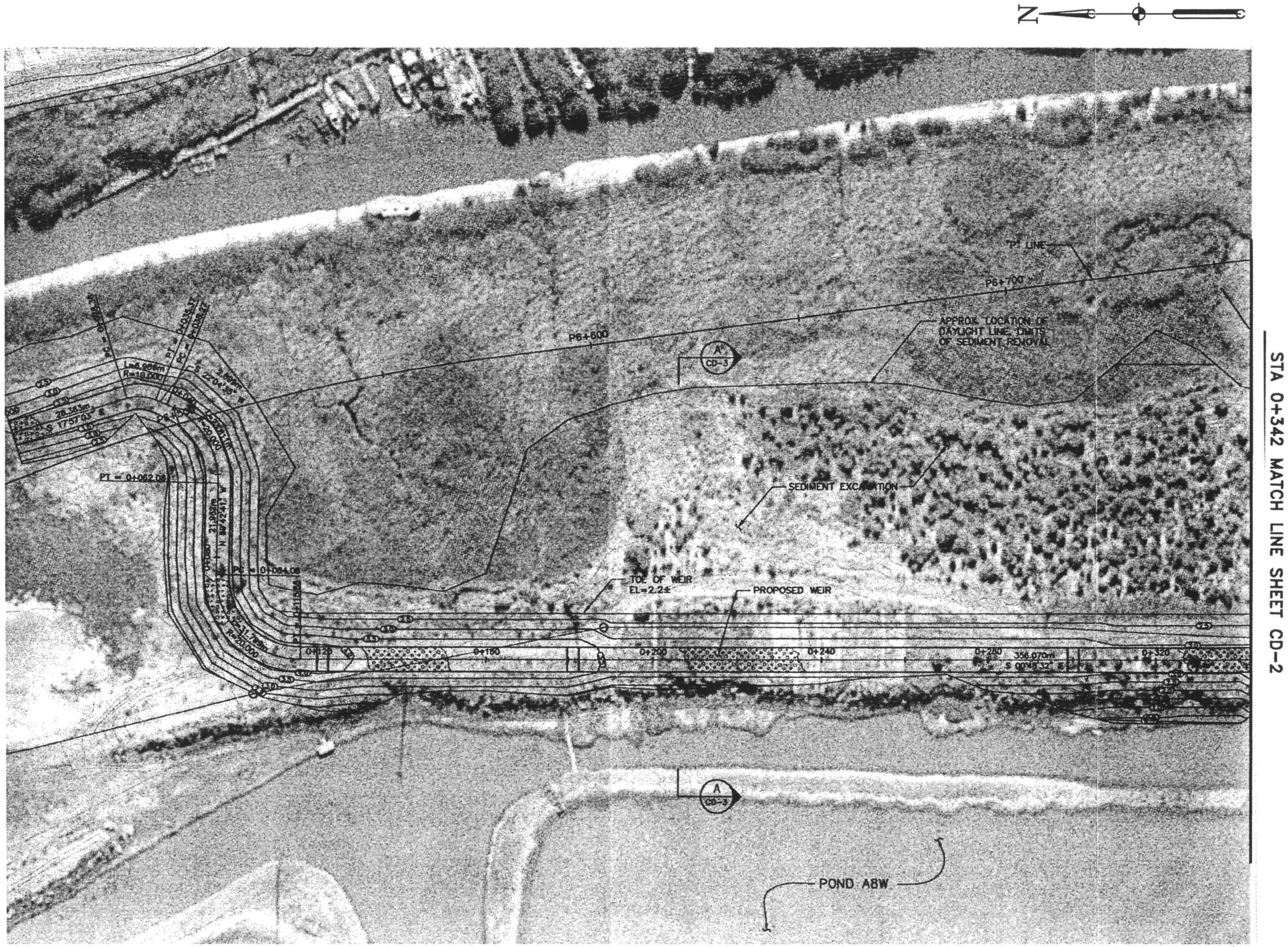
REVISION 5

PROJECT 95-00112030.00

DRAWING Fig. 2

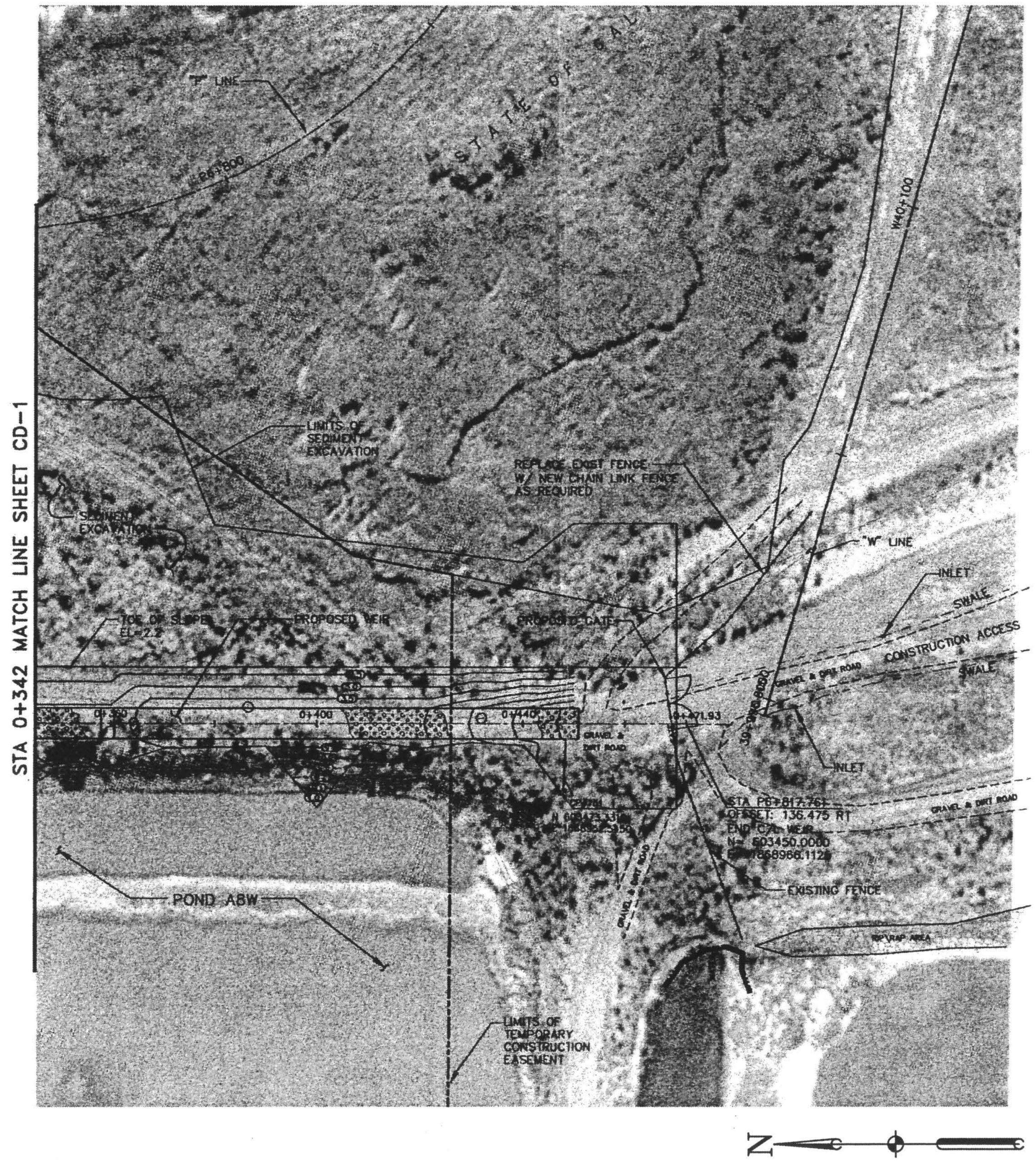
SHEET 1 OF 1

TEMP013



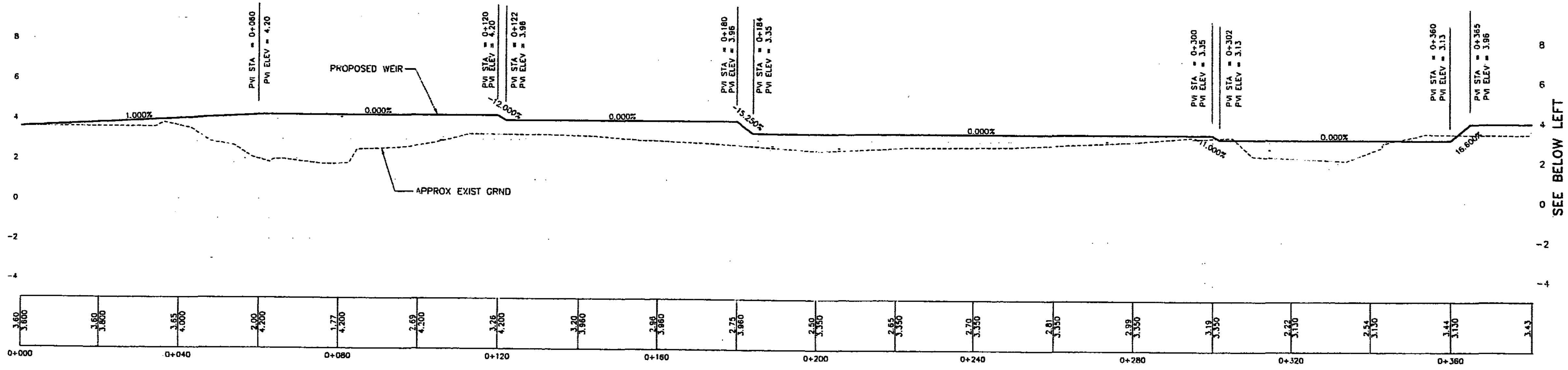
q:\cadd2\01files\95-00112030.00\246\weir3.dwg

| | | | |
|---|---|---|---|
| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | **PLAN**<br>**WEST LEVEE OVERFLOW WEIR** | Figure<br>3 |
| TEMPOO4 7/9/02 | **URS** | | |



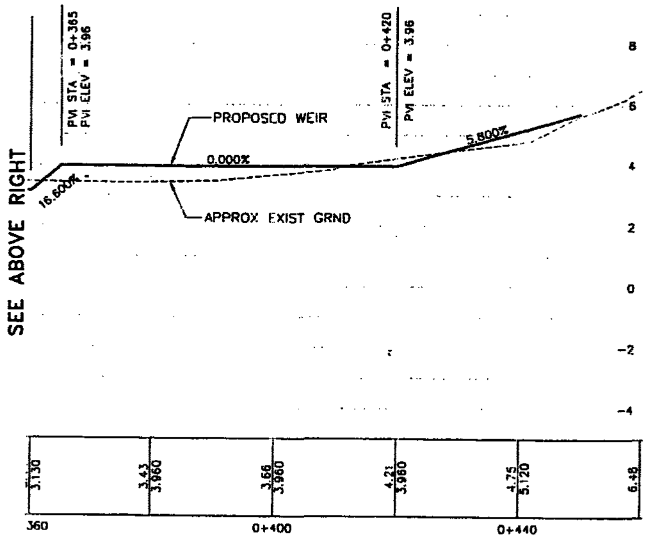
q:\cadd2\01files\95-00112030.00\246\weir4.dwg

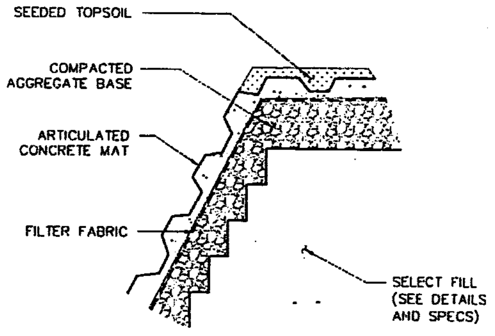
| | | | |
|---|---|---|---|
| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | **PLAN**<br>**WEST LEVEE OVERFLOW WEIR** | Figure<br>4 |
| TEMP004 7/9/02 | | | |
| | **URS** | | |



## PROFILE ALONG WEST LEVEE OVERFLOW WEIR

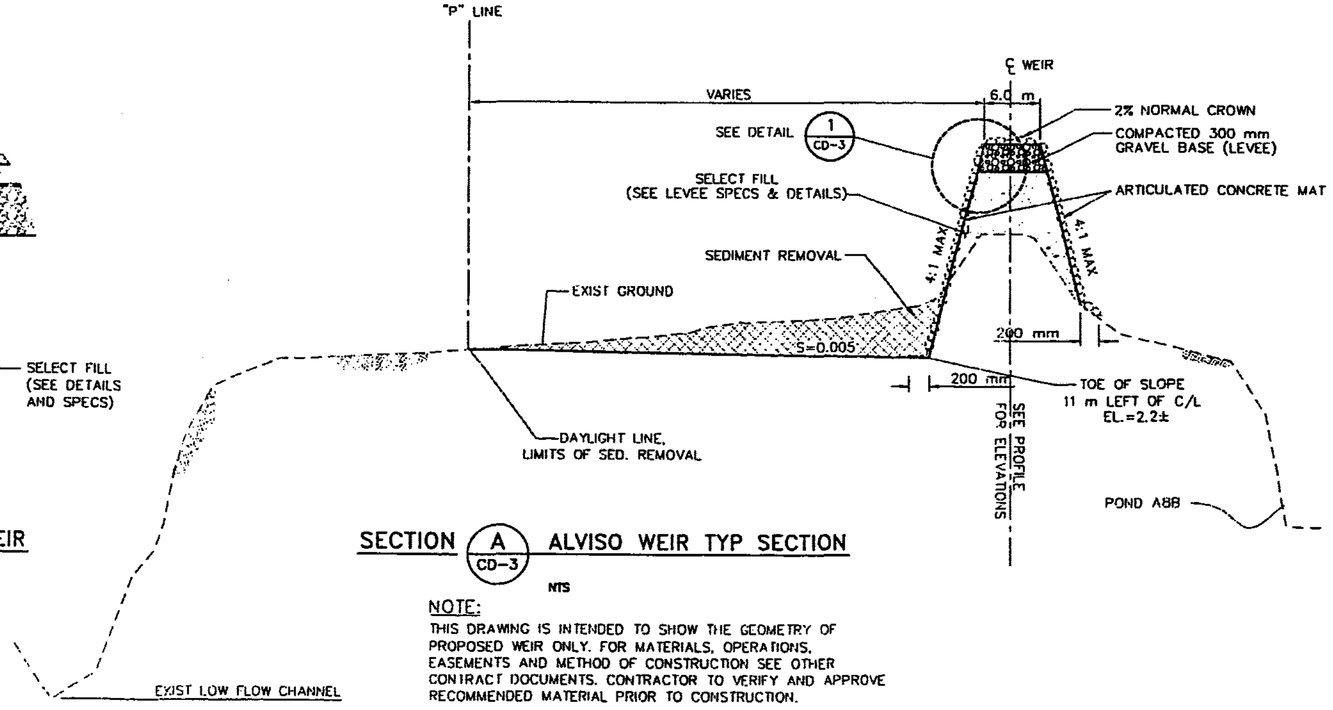
SCALE: H 1:500
V 1:100





DETAIL (1 / CD-3) ALVISO WEIR

NTS



SECTION (A / CD-3) ALVISO WEIR TYP SECTION

NTS

NOTE:
THIS DRAWING IS INTENDED TO SHOW THE GEOMETRY OF PROPOSED WEIR ONLY. FOR MATERIALS, OPERATIONS, EASEMENTS AND METHOD OF CONSTRUCTION SEE OTHER CONTRACT DOCUMENTS. CONTRACTOR TO VERIFY AND APPROVE RECOMMENDED MATERIAL PRIOR TO CONSTRUCTION.

q:\cadd2\01files\95-00112030.00\246\weir5.dwg

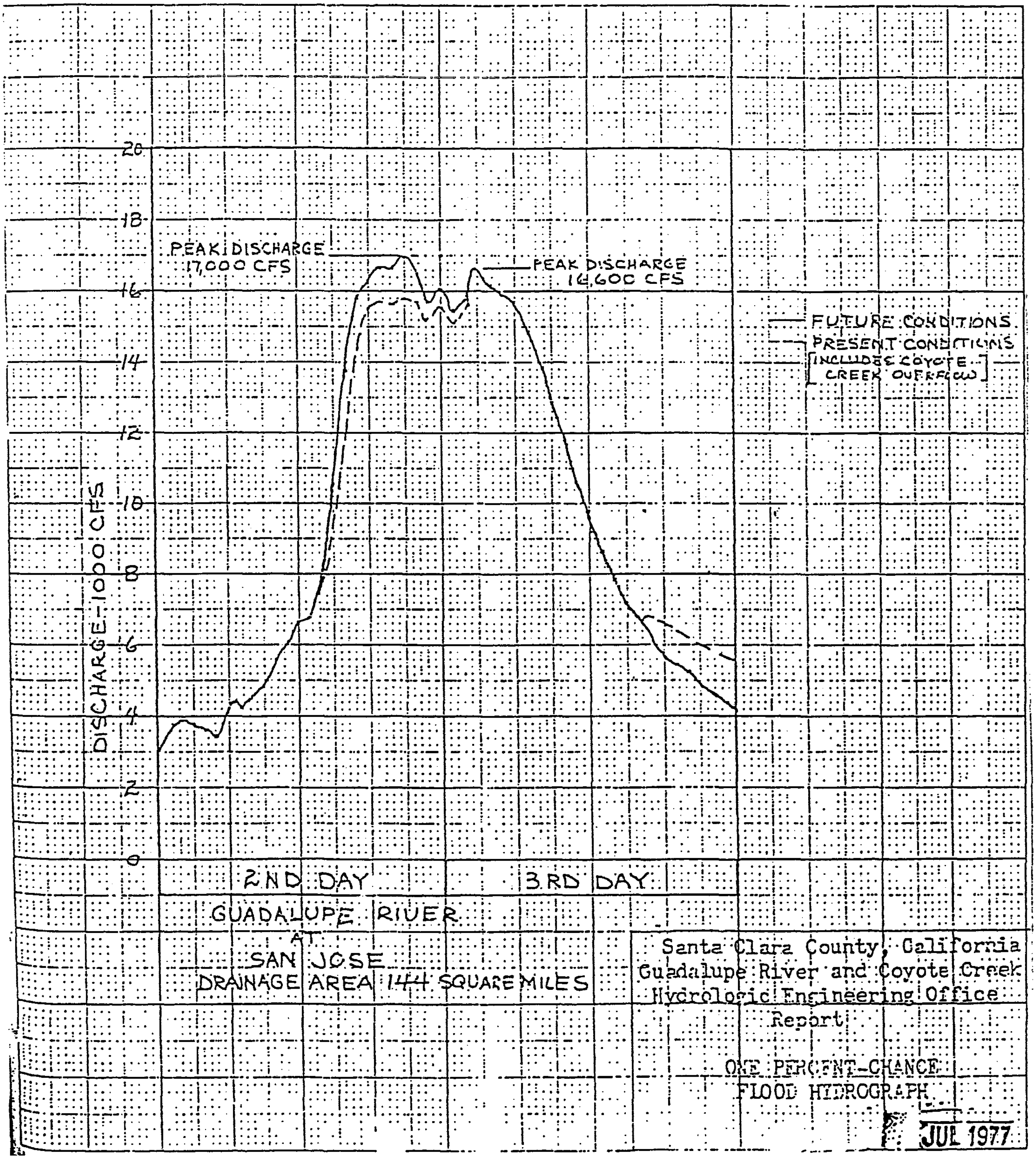
Project No.
95-00112030.00
7/9/02

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara County, California

URS

**PROFILE AND TYPICAL SECTIONS
WEST LEVEE OVERFLOW WEIR**

Figure
5



Q:\CADD2\01FILES\95-00112030.00\246\GUADFIG06.DWG

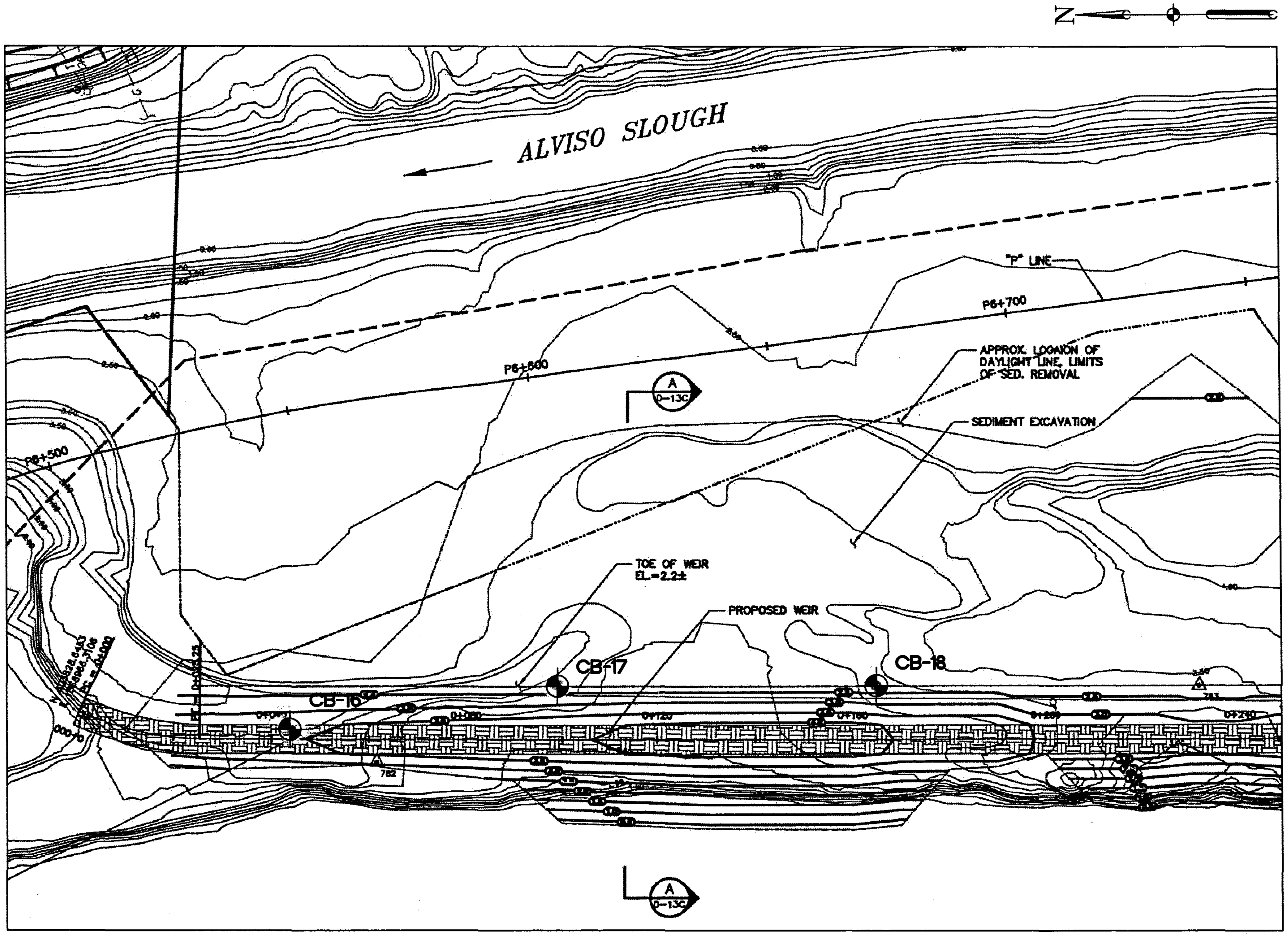
Project No.
95-00112030.00
5/31/02

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara County, California

URS

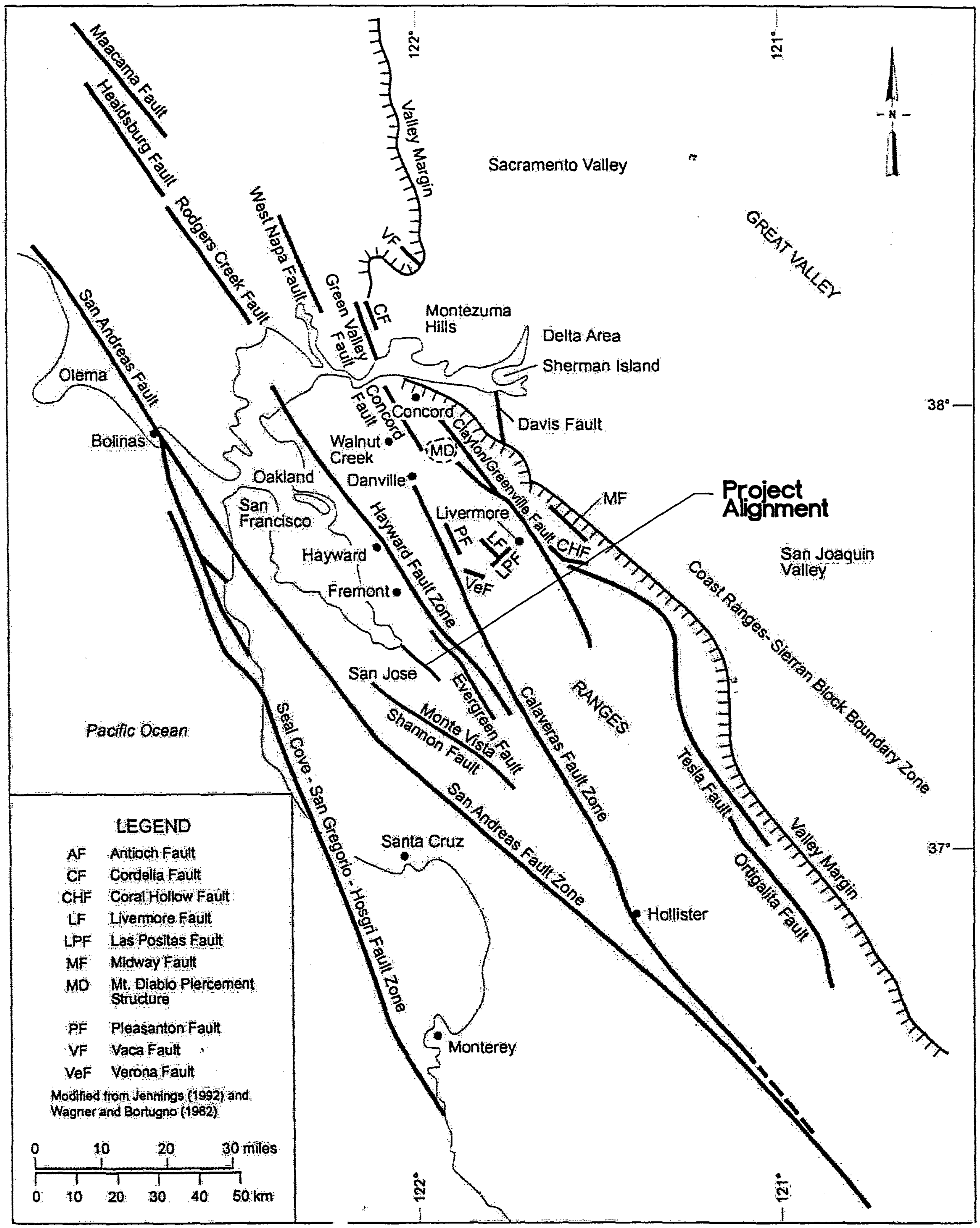
## ONE PERCENT CHANCE FLOOD HYDROGRAPH

Figure
6



q:\cadd2\01files\95-00112030.00\246\BORLOCMAP.dwg

| Project No. | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | BORING LOCATION PLAN<br>WEST LEVEE OVERFLOW WEIR | Figure<br>7 |
|---|---|---|---|
| 95-00112030.00<br>5/31/02 | URS | | |

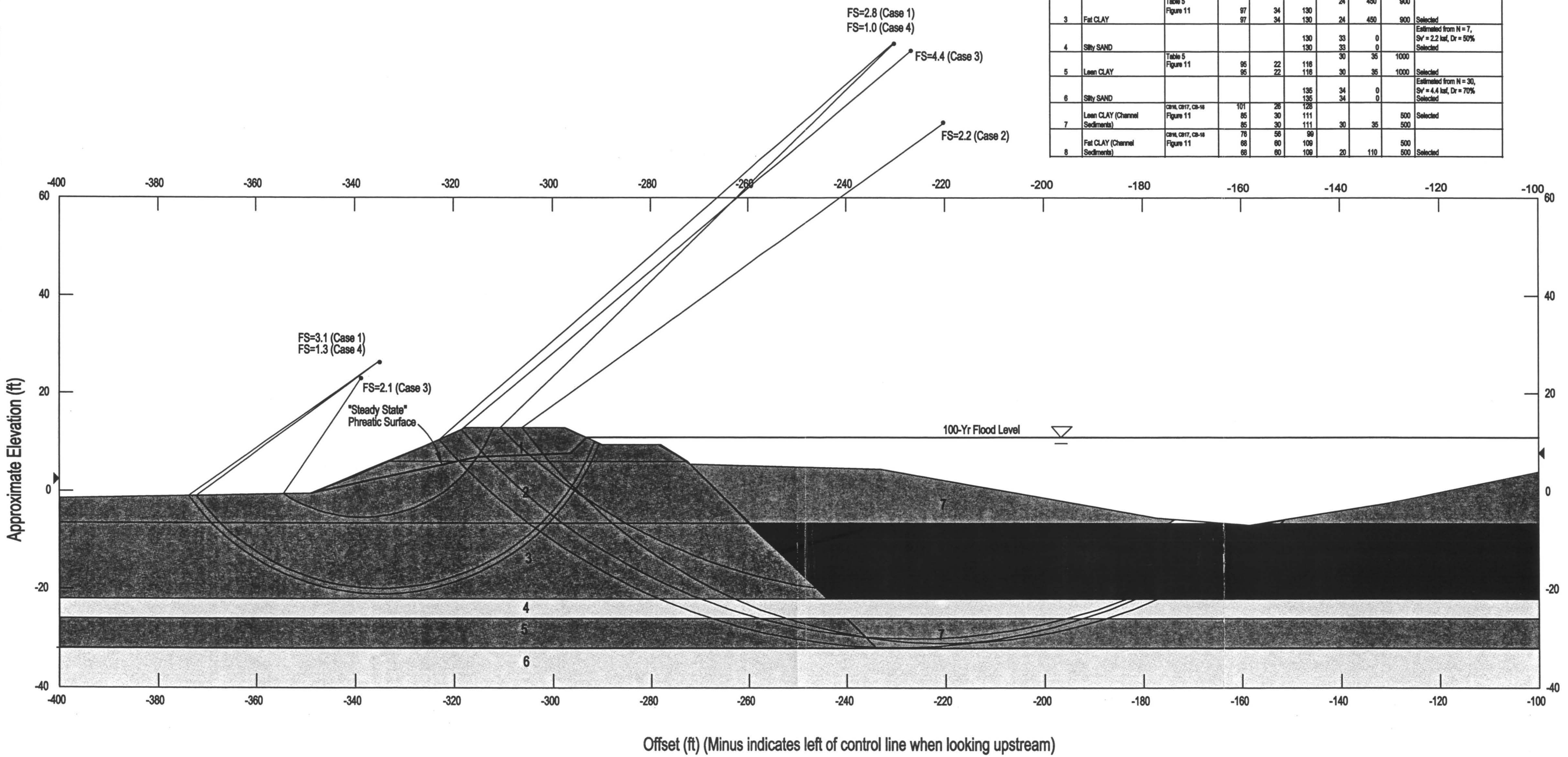


q:\cadd2\01files\95-00112030.00\246\faultmap08.dwg

| Project No. 95-00112030.00 | LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT<br>Santa Clara County, California | REGIONAL ACTIVE FAULTS | Figure 8 |
|---|---|---|---|
| 5/31/02 | | | |
| URS | | | |

Summary of Geotechnical Design Parameters

| Layer | Soil Type | Reference | Dry Density (pcf) | Moisture Content (%) | Total Unit Weight (pcf) | ϕ' (deg) | c' (psf) | Su (psf) | Remarks |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Lean Clay (FILL) | Table 5 | | | | 24 | 450 | | |
| | | Figure 11 | 100 | 20 | 120 | | | | |
| | | | 100 | 20 | 120 | 24 | 450 | 1500 | Selected |
| 2 | Lean CLAY | Table 5 | | | | 30 | 150 | | |
| | | Figure 11 | 102 | 24 | 126 | | | 750 | |
| | | | 102 | 24 | 126 | 30 | 150 | 750 | Selected |
| 3 | Fat CLAY | Table 5 | | | | 24 | 450 | 900 | |
| | | Figure 11 | 97 | 34 | 130 | | | | |
| | | | 97 | 34 | 130 | 24 | 450 | 900 | Selected |
| 4 | Silty SAND | | | | | | | | Estimated from N = 7, |
| | | | | | 130 | 33 | 0 | | Sv' = 2.2 ksf, Dr = 50% |
| | | | | | 130 | 33 | 0 | | Selected |
| 5 | Lean CLAY | Table 5 | | | | 30 | 35 | 1000 | |
| | | Figure 11 | 95 | 22 | 116 | | | | |
| | | | 95 | 22 | 116 | 30 | 35 | 1000 | Selected |
| 6 | Silty SAND | | | | | | | | Estimated from N = 30, |
| | | | | | 135 | 34 | 0 | | Sv' = 4.4 ksf, Dr = 70% |
| | | | | | 135 | 34 | 0 | | Selected |
| 7 | Lean CLAY (Channel Sediments) | CB16, CB17, CB-18 | 101 | 26 | 128 | | | | |
| | | Figure 11 | 85 | 30 | 111 | | | 500 | Selected |
| | | | 85 | 30 | 111 | 30 | 35 | 500 | |
| 8 | Fat CLAY (Channel Sediments) | CB16, CB17, CB-18 | 76 | 56 | 99 | | | | |
| | | Figure 11 | 68 | 60 | 109 | | | 500 | |
| | | | 68 | 60 | 109 | 20 | 110 | 500 | Selected |



q:\cadd2\01files\95-00112030.00\246\crosssec31.dwg

Project No.
95-00112030.00
TEMP004
9/23/02

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara County, California

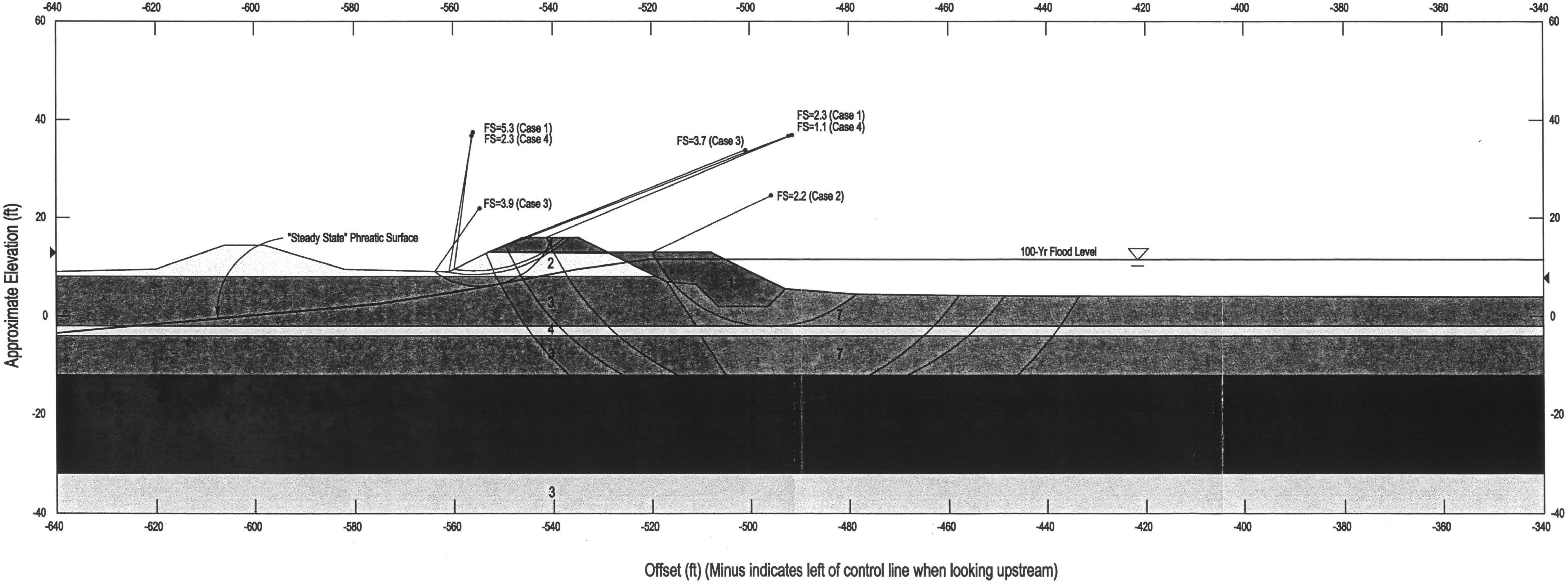
URS

**STABILITY ANALYSIS**
**CROSS SECTION 31**
**STATION 6+300 EAST LEVEE**

Figure
9

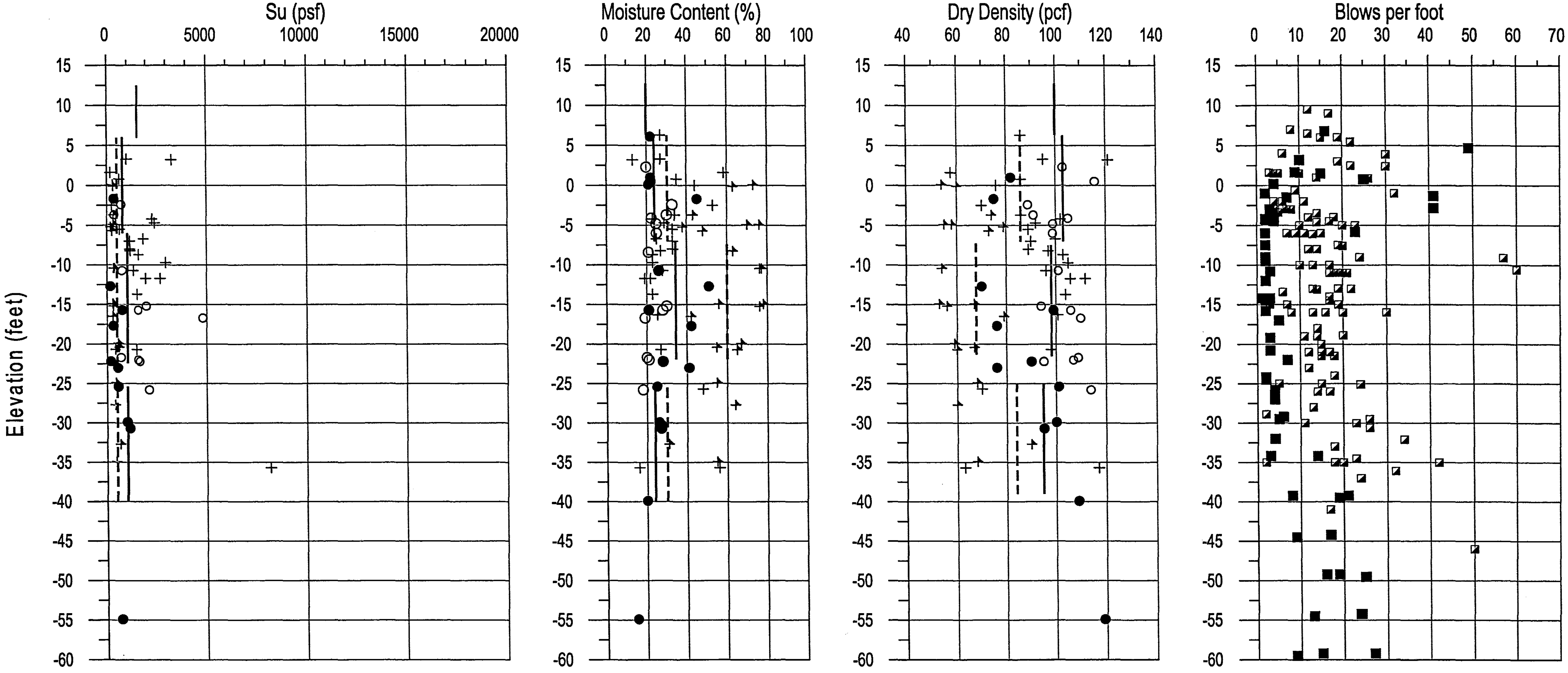
Summary of Geotechnical Design Parameters

| Layer | Soil Type | Reference | Dry Density (pcf) | Moisture Content (%) | Total Unit Weight (pcf) | ϕ' (deg) | c' (psf) | Su (psf) | Remarks |
|---|---|---|---|---|---|---|---|---|---|
| | | Table 5 | | | | 24 | 450 | | |
| | | Figure 12 | 109 | 20 | 131 | | | 1500 | |
| 1 | Lean CLAY (FILL) | EB27, EB28 | 100 | 26 | 126 | 24 | 450 | 1500 | Selected |
| | | | | | | | | | Estimated from N = 10, |
| | | | | | | 33 | 0 | | σv' = 0.3 ksf, Dr = 70% |
| 2 | Silty SAND (FILL) | | | | 135 | 33 | 0 | | Selected |
| | | Table 5 | | | | 30 | 35 | | |
| | | Figure 12 | 100 | 30 | 130 | | | 1000 | |
| 3 | Lean CLAY | EB27, EB28 | 99 | 30 | 129 | 30 | 35 | 1000 | Selected |
| | | | | | | | | | Estimated from N = 5, |
| | | | | | | 33 | 0 | | σv' = 1.9 ksf, Dr = 40% |
| 4 | Silty SAND | | | | 135 | 33 | 0 | | Selected |
| | | Table 5 | | | | 20 | 110 | | |
| | | Figure 12 | 75 | 55 | 116 | | | 500 | |
| 5 | Fat CLAY | EB27, EB28 | 70 | 52 | 106 | 20 | 110 | 500 | Selected |
| 6 | Fat CLAY (Channel Sediments) | CB01, CB03, CB14, CB15 | 80 | 45 | 116 | 20 | 110 | 400 | Selected |
| 7 | Lean CLAY (Channel Sediments) | CB01, CB03, CB14, CB15 | 90 | 30 | 117 | 30 | 150 | 425 | Selected |



q:\cadd2\01files\95-00112030.00\246\crossec32.dwg

| | | | |
|---|---|---|---|
| Project No.<br>95-00112030.00<br>9/23/02 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | **STABILITY ANALYSIS<br>CROSS SECTION 32<br>STATION 6+780 EAST LEVEE** | Figure<br>10 |
| **URS** | | | |



Data from the following borings and CPTs:

GJJ-2
GJJ-3
GRIM-10
GRIM-11
GRIM-2

GRIM-3
GRIM-4
GRIM-9
PRMAR-5
PYH-3

REAL1-2
SBYC-16
EB-01
EB-27
EB-28

CB-14
CB-15
CB-16
CB-17
CB-18

| Levee Fill and Foundation Soil | Channel Sediments |
|---|---|
| + Fat Clay | + Fat Clay |
| ○ Lean Clay | ● Lean Clay |
| ◪ All Soil Types | ■ All Soil Types |

LEGEND:

| SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

¦ SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\246\index31.dwg

Project No.
95-00112030.00

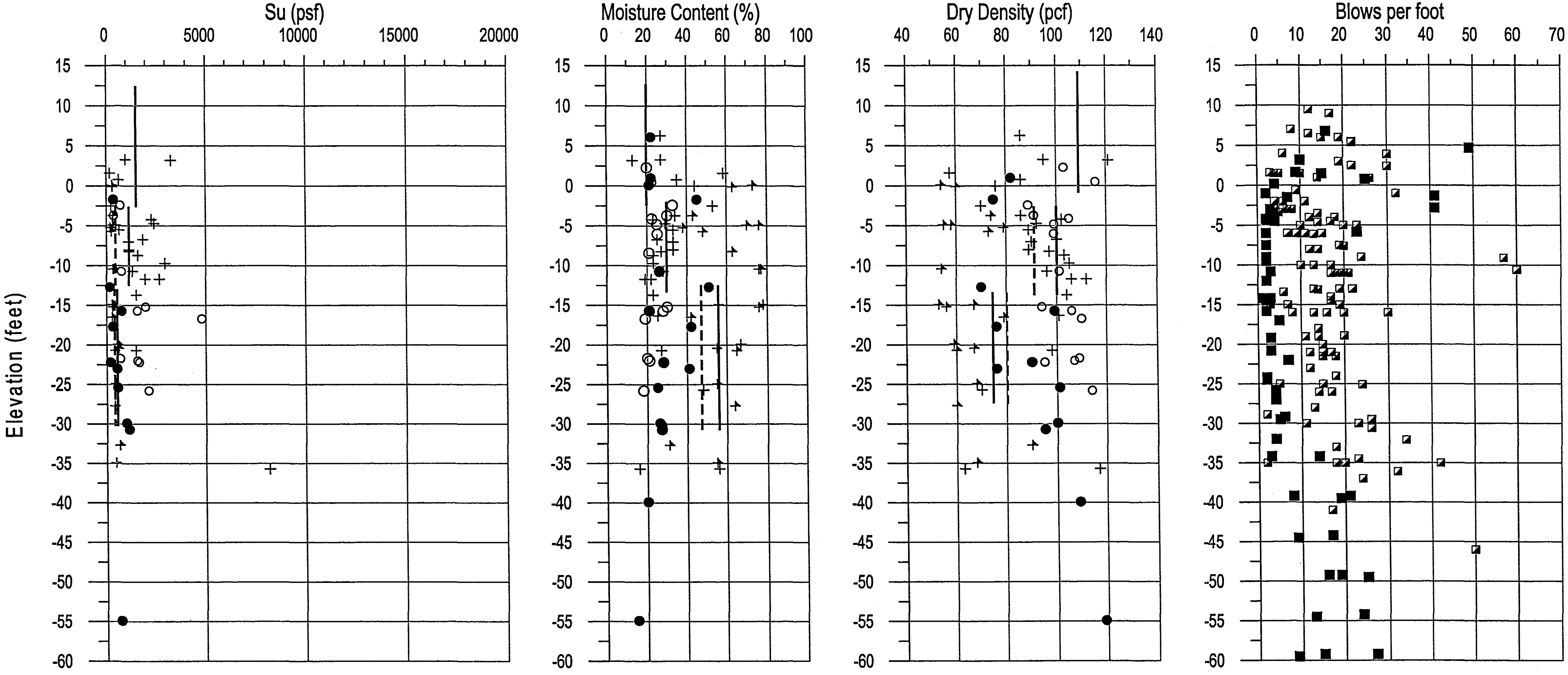
6/14/02

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara County, California

**URS**

## INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 31

Figure
11



Data from the following borings and CPTs:

GJJ-2
GJJ-3
GRIM-10
GRIM-11
GRIM-2

GRIM-3
GRIM-4
GRIM-9
PRMAR-5
PYH-3

REAL1-2
SBYC-16
EB-01
EB-27
EB-28

CB-14
CB-15
CB-16
CB-17
CB-18

| Levee Fill and Foundation Soil | Channel Sediments |
|---|---|
| + Fat Clay | + Fat Clay |
| ○ Lean Clay | ● Lean Clay |
| □ All Soil Types | ◪ All Soil Types |

LEGEND:

| SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

¦ SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\246\index32.dwg

| Project No. 95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | INDEX PROPERTY PLOTS FOR<br>CROSS SECTION NO. 32 | Figure<br>12 |
|---|---|---|---|
| 6/14/02 | URS | | |

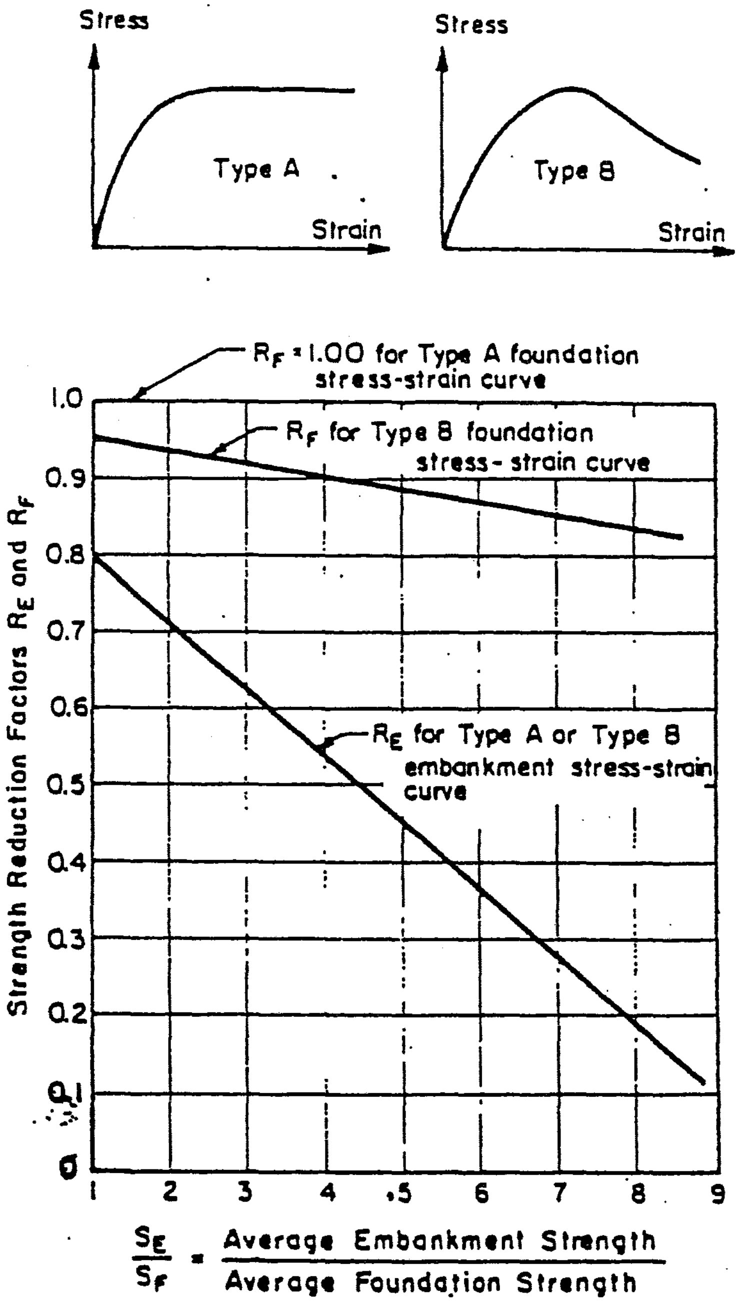


Fig. 3 CORRECTION FACTORS $R_E$ AND $R_F$ TO ACCOUNT FOR PROGRESSIVE FAILURE IN EMBANKMENTS ON SOFT CLAY FOUNDATIONS.
(after Chirapuntu and Duncan, 1975)

Q:\CADD2\01FILES\95-00112030.00\246\guadFig13.dwg

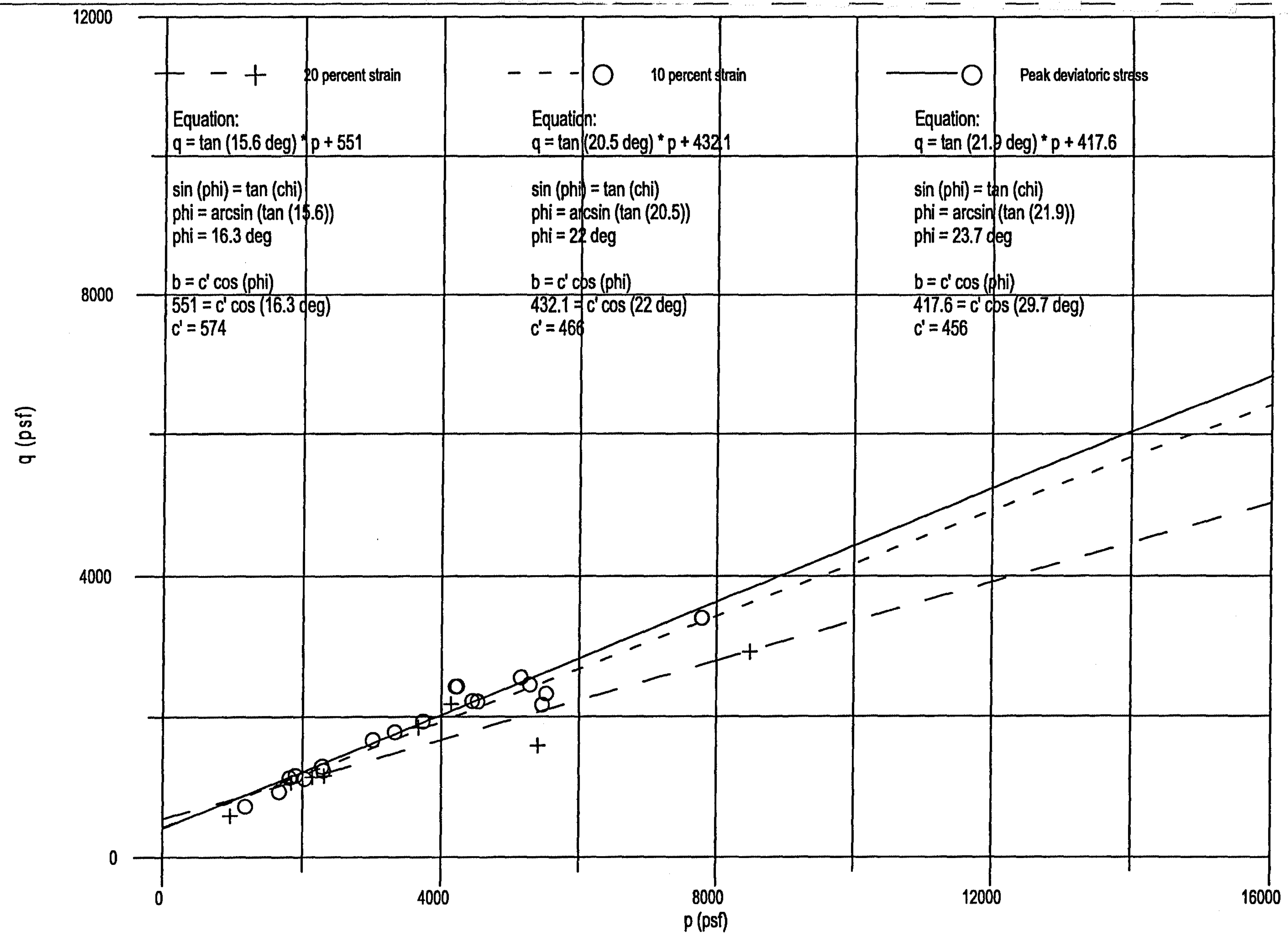
Project No.
95-00112030.00
5/31/02

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara County, California

URS

# EMBANKMENT STRENGTH REDUCTION FACTOR FOR SOFT CLAY FOUNDATIONS

Figure
13



Equation:
q = tan (15.6 deg) * p + 551

sin (phi) = tan (chi)
phi = arcsin (tan (15.6))
phi = 16.3 deg

b = c' cos (phi)
551 = c' cos (16.3 deg)
c' = 574

Equation:
q = tan (20.5 deg) * p + 432.1

sin (phi) = tan (chi)
phi = arcsin (tan (20.5))
phi = 22 deg

b = c' cos (phi)
432.1 = c' cos (22 deg)
c' = 466

Equation:
q = tan (21.9 deg) * p + 417.6

sin (phi) = tan (chi)
phi = arcsin (tan (21.9))
phi = 23.7 deg

b = c' cos (phi)
417.6 = c' cos (29.7 deg)
c' = 456

Q:\CADD2\01FILES\95-00112030.00\246\guadFig14.dwg

Project No.
95-00112030.00

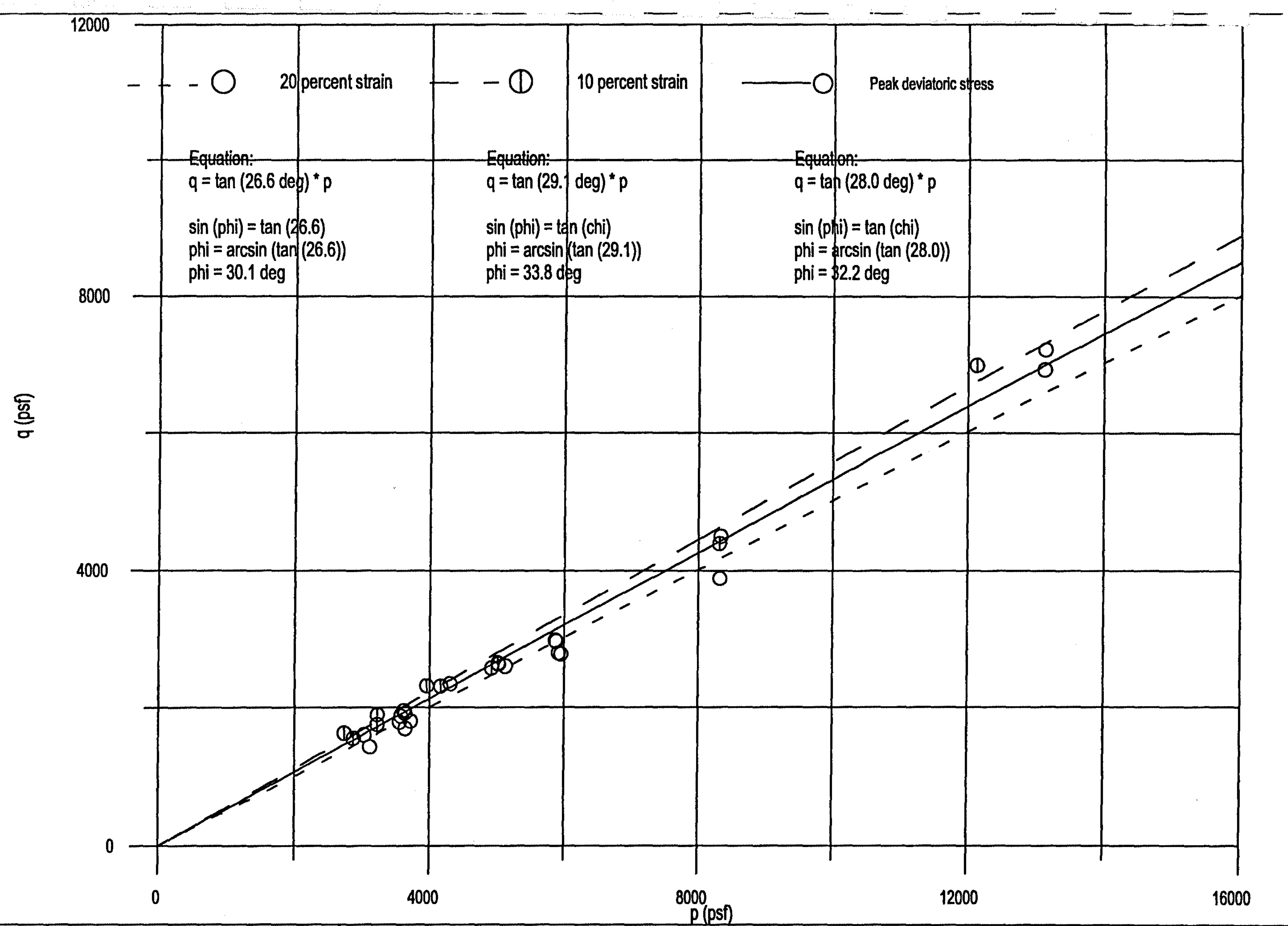
7/15/02

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara County, California

**URS**

## Shear Strength Parameters
## For
## Lean Clay Fill with Dry Density less than 110 pcf

Figure
14



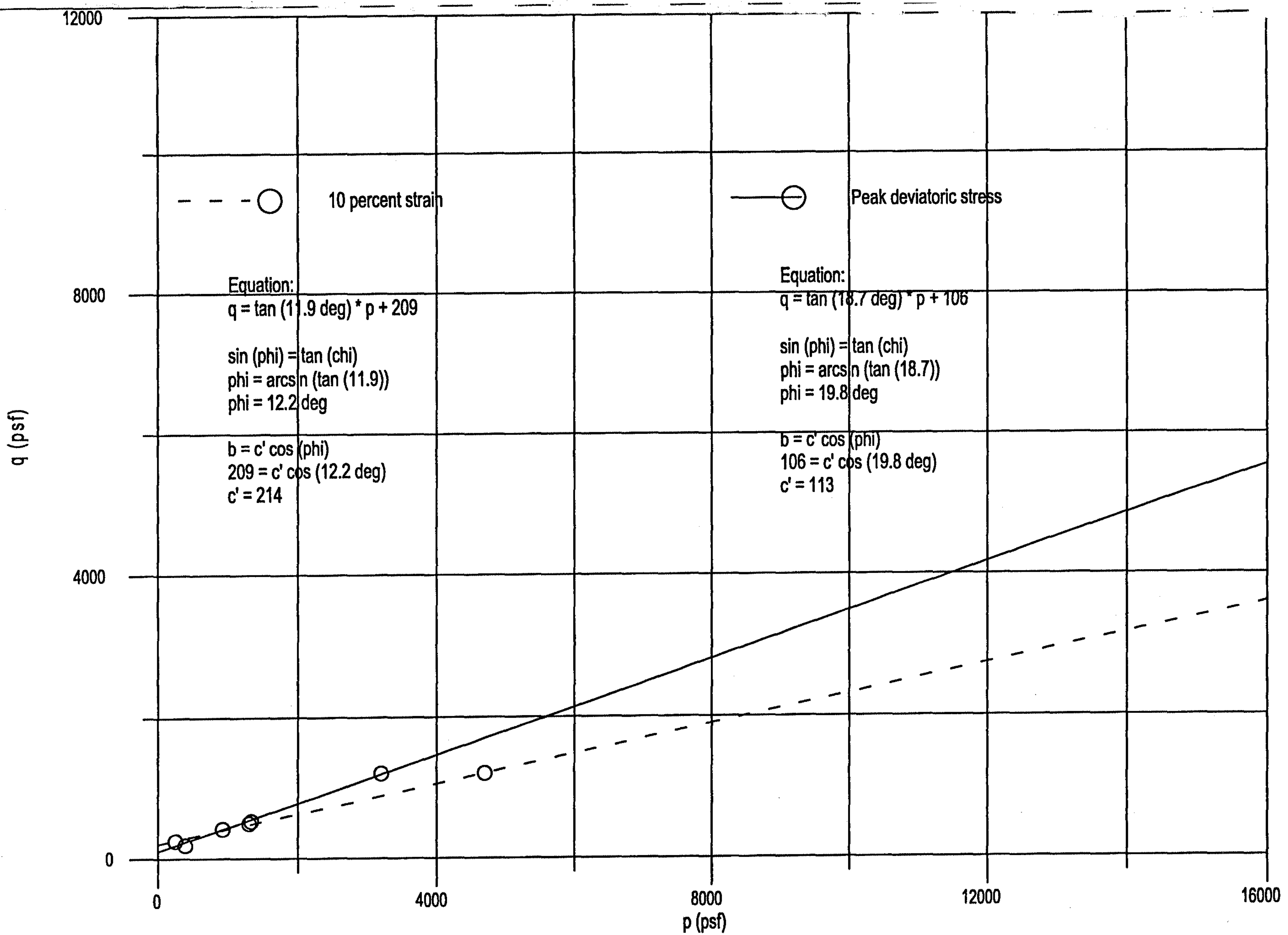
Q:\CADD2\01FILES\95-00112030.00\246\guadFig15.dwg

| | |
|---|---|
| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California |
| 7/15/02 | |

**URS**

**Shear Strength Parameters**
**For**
**Lean Clay Fill with Dry Density greater than 110 pcf**

Figure
15



Equation:
q = tan (11.9 deg) * p + 209

sin (phi) = tan (chi)
phi = arcsin (tan (11.9))
phi = 12.2 deg

b = c' cos (phi)
209 = c' cos (12.2 deg)
c' = 214

Equation:
q = tan (18.7 deg) * p + 106

sin (phi) = tan (chi)
phi = arcsin (tan (18.7))
phi = 19.8 deg

b = c' cos (phi)
106 = c' cos (19.8 deg)
c' = 113

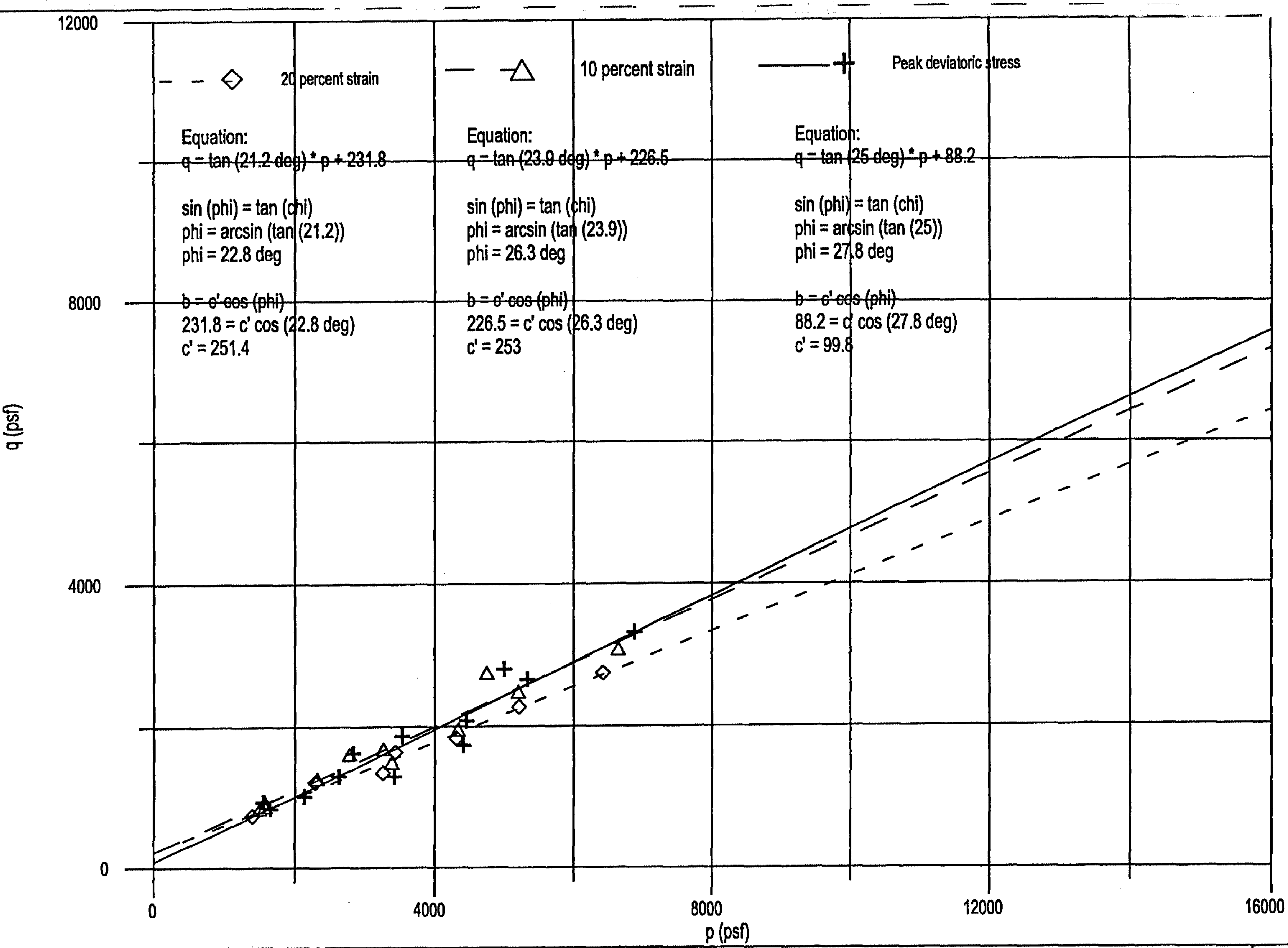
Q:\CADD2\01FILES\95-00112030.00\246\guadFig16.dwg

| | |
|---|---|
| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California |
| 7/15/02 | |

URS

## Shear Strength Parameters For Fat Clay with Dry Density less than 80 pcf

Figure 16



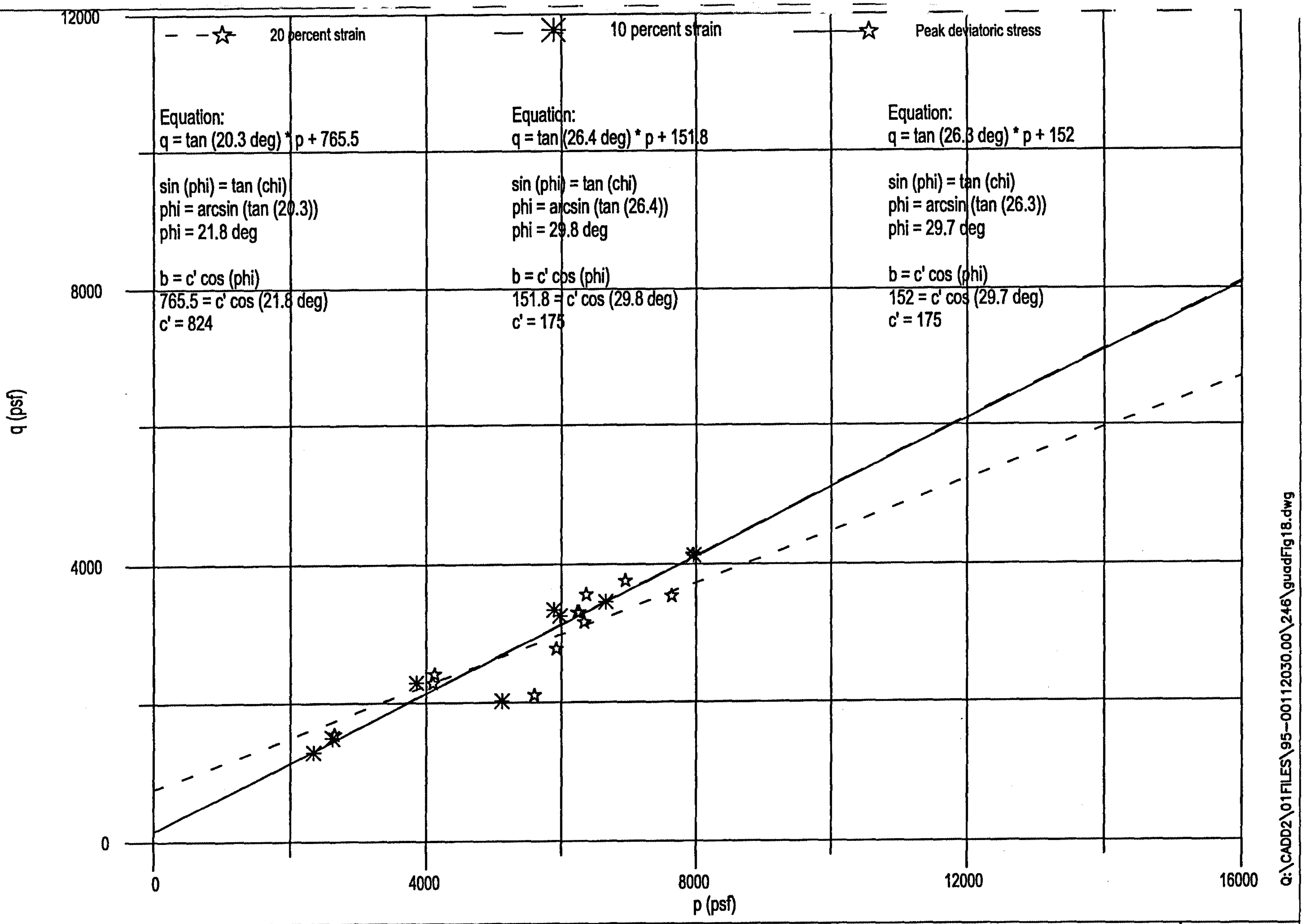
q:\CADD2\01FILES\95-00112030.00\246\guadFig17.dwg

| | |
|---|---|
| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California |
| 7/15/02 | |

**URS**

## Shear Strength Parameters For Lean Clay with Dry Density between 80 pcf and 100 pcf

Figure 17



Q:\CADD2\01FILES\95-00112030.00\246\guadFig18.dwg

Project No.
95-00112030.00

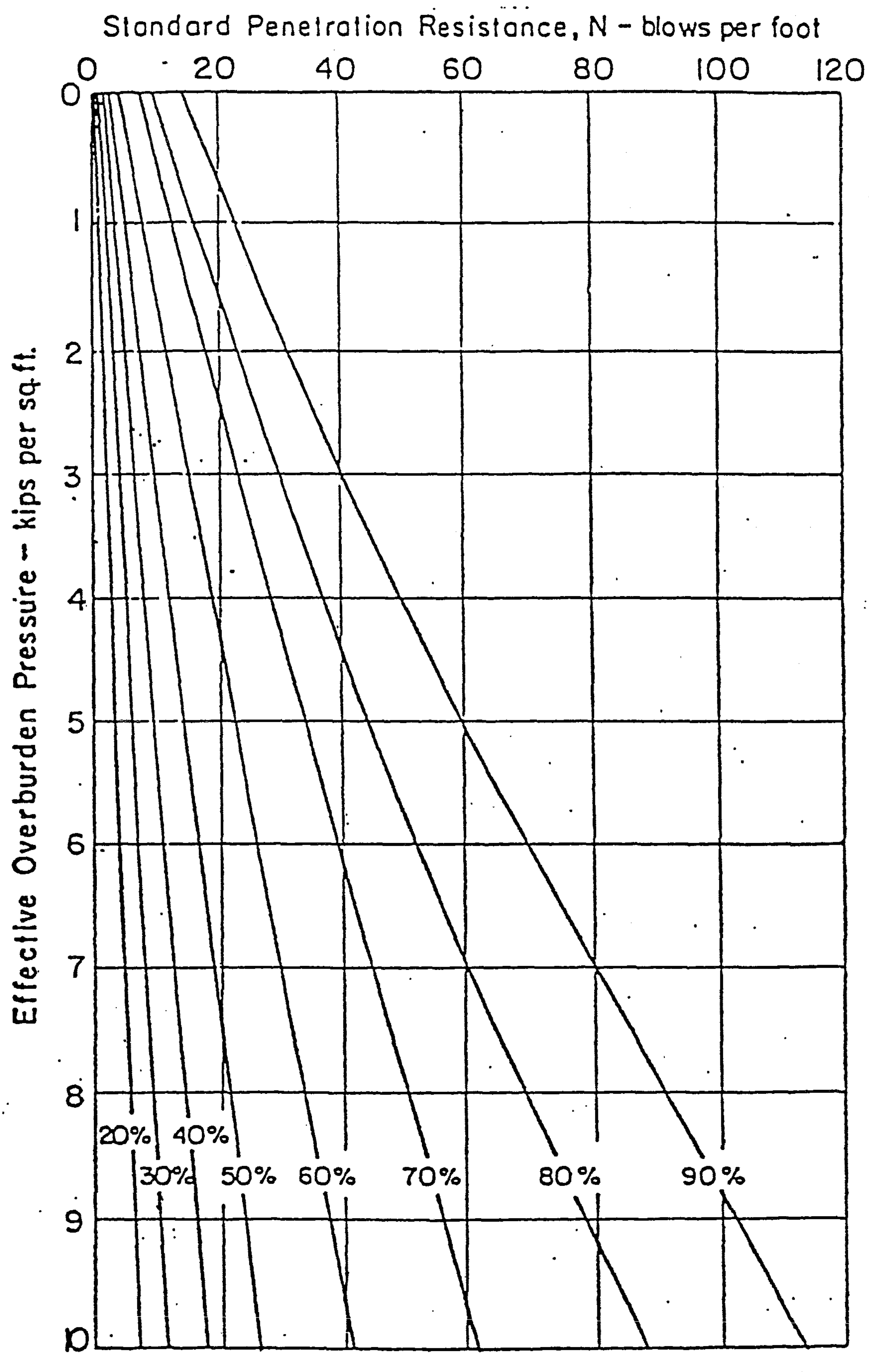
7/15/02

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara County, California

**URS**

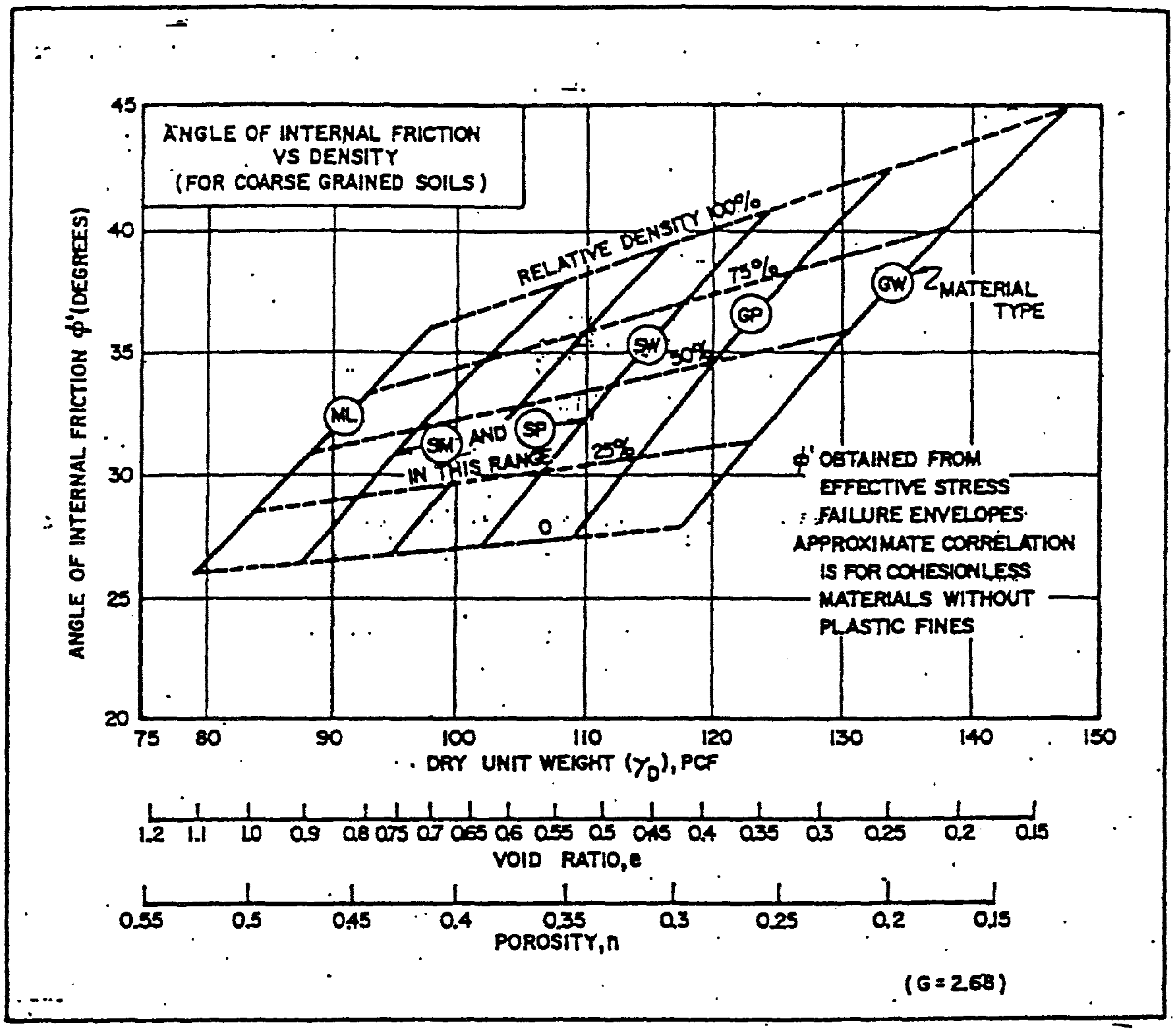
## Shear Strength Parameters For Lean Clay with Dry Density between 100 pcf and 110 pcf

Figure
18



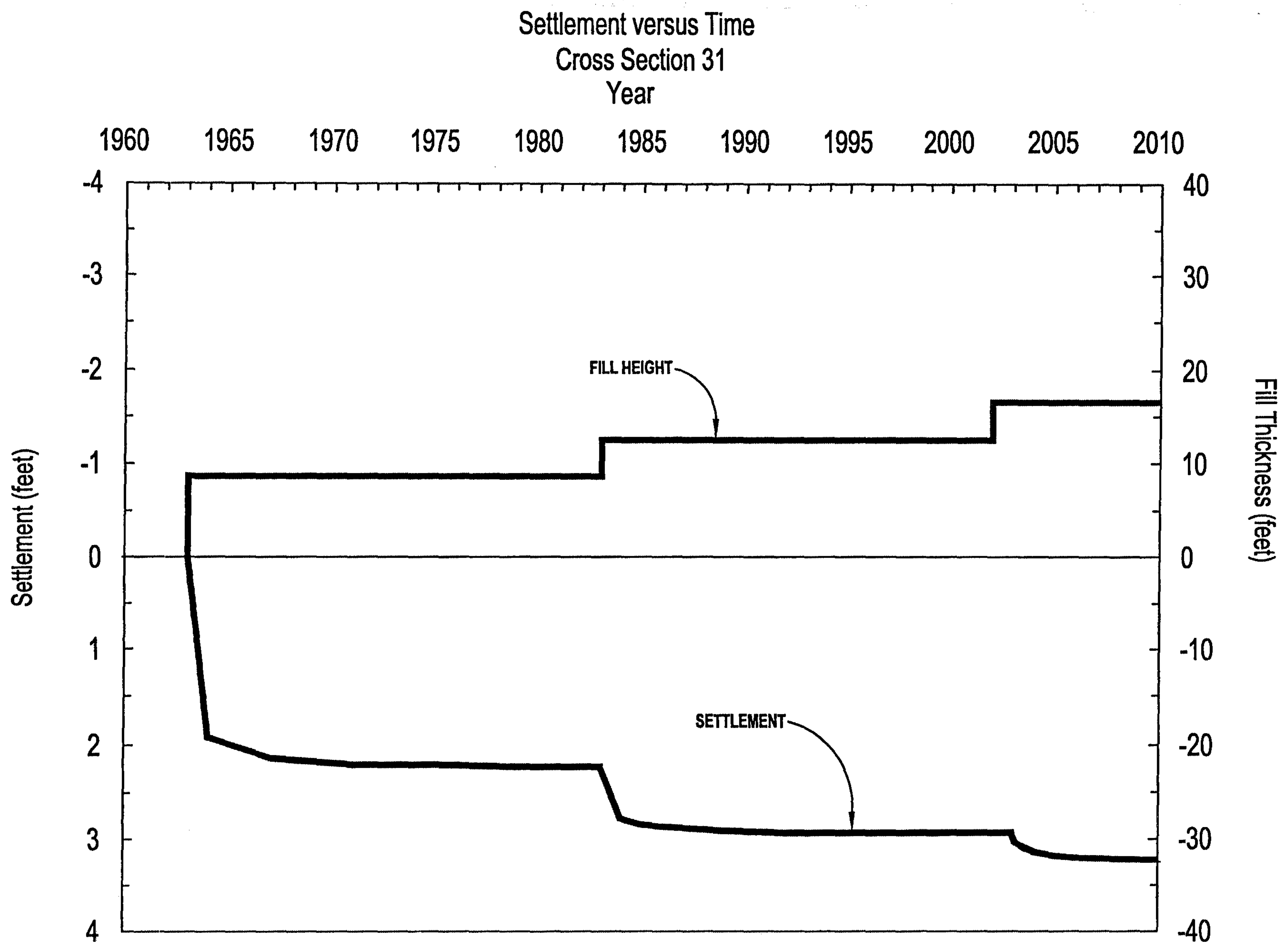
| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | **Relationship between Standard Penetration Resistance, Relative Density and Effective Overburden Pressure**<br>(AFTER GIBBS AND HOLTZ, 1957) | Figure<br>19 |
|---|---|---|---|
| 7/15/02 | | | |
| URS | | | |

Q:\CADD2\01FILES\95-00112030.00\246\guadFig19.dwg



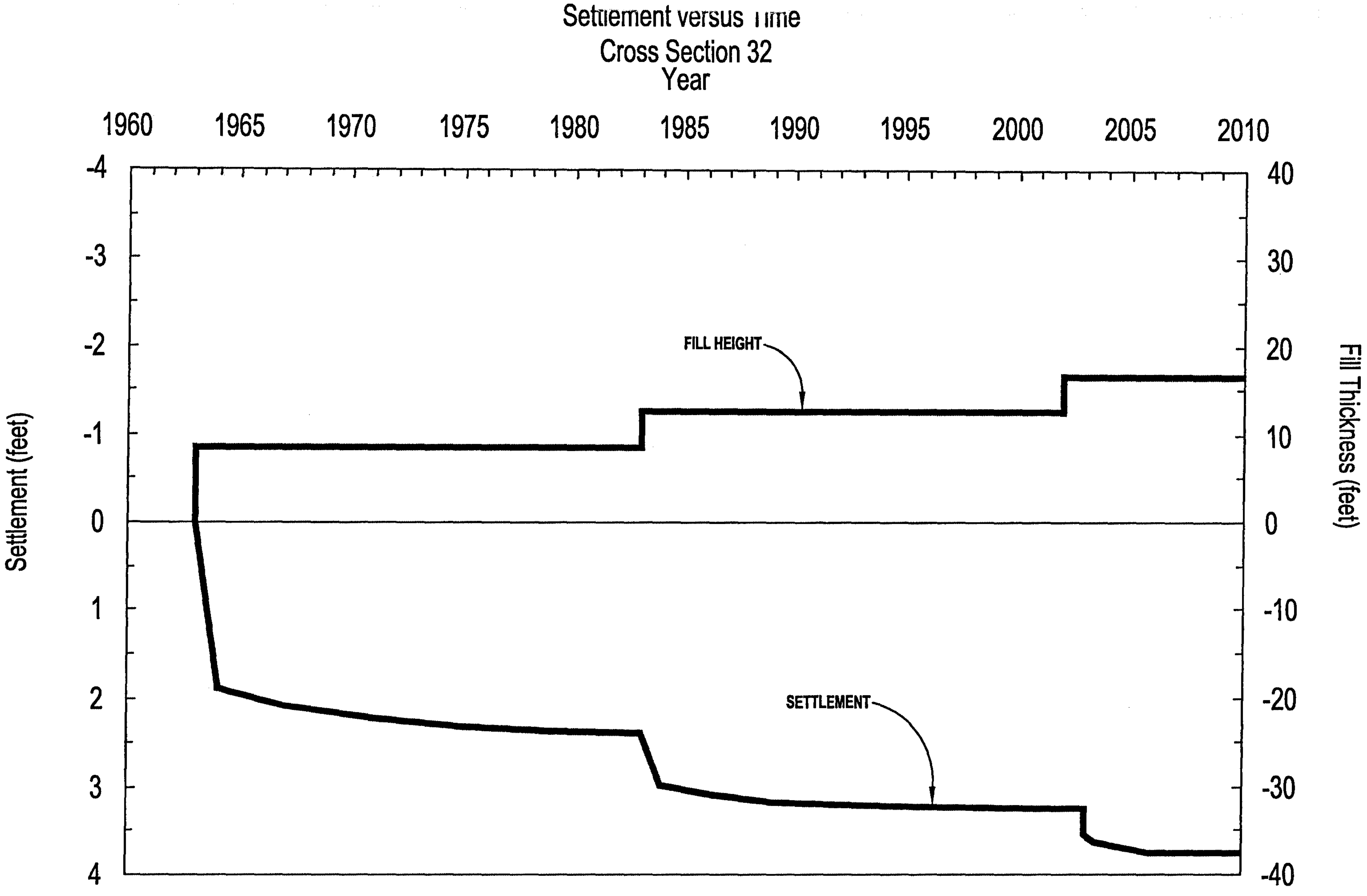
Q:\CADD2\01FILES\95-00112030.00\246\guadFig20.dwg

| | | | |
|---|---|---|---|
| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | **Correlation of Strength Characteristics for Granular Soils (NAVFAC, 1986)** | Figure<br>20 |
| 7/15/02 | | | |
| **URS** | | | |



Q:\CADD2\01FILES\95-00112030.00\246\timeratesec31.dwg

| | | |
|---|---|---|
| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | **Estimated Time Rate of Settlement for Cross Section 31** |
| 5/31/02 | | Figure<br>21 |
| URS | | |



q:\CADD2\01FILES\95-00112030.00\246\timeratesec32.dwg

| | | | |
|---|---|---|---|
| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | **Estimated Time Rate of Settlement for Cross Section 32** | Figure<br>22 |
| 5/31/02 | | | |
| URS | | | |

<table>
<tr><td></td><td colspan="12">Coefficient of Permeability $k$ in cm per sec (log scale)</td></tr>
<tr><td></td><td>$10^{2}$</td><td>$10^{1}$</td><td>1.0</td><td>$10^{-1}$</td><td>$10^{-2}$</td><td>$10^{-3}$</td><td>$10^{-4}$</td><td>$10^{-5}$</td><td>$10^{-6}$</td><td>$10^{-7}$</td><td>$10^{-8}$</td><td>$10^{-9}$</td></tr>
<tr><td>Drainage</td><td colspan="6">Good</td><td colspan="2">Poor</td><td colspan="4">Practically Impervious</td></tr>
<tr><td rowspan="2">Soil types</td><td colspan="2" rowspan="2">Clean gravel</td><td colspan="3">Clean sands, clean sand and gravel mixtures</td><td colspan="4">Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits, etc.</td><td colspan="3" rowspan="2">"Impervious" soils, e.g., homogeneous clays below zone of weathering</td></tr>
<tr><td colspan="2"></td><td colspan="5">"Impervious" soils modified by effects of vegetation and weathering</td></tr>
<tr><td rowspan="2">Direct determination of $k$</td><td colspan="6">Direct testing of soil in its original position—pumping tests. Reliable if properly conducted. Considerable experience required</td><td colspan="6" rowspan="2"></td></tr>
<tr><td colspan="5">Constant-head permeameter. Little experience required</td><td></td></tr>
<tr><td rowspan="2">Indirect determination of $k$</td><td colspan="2"></td><td colspan="3">Falling-head permeameter. Reliable. Little experience required</td><td colspan="3">Falling-head permeameter. Unreliable. Much experience required</td><td colspan="4">Falling-head permeameter. Fairly reliable. Considerable experience necessary</td></tr>
<tr><td colspan="5">Computation from grain-size distribution. Applicable only to clean cohesionless sands and gravels</td><td colspan="4"></td><td colspan="3">Computation based on results of consolidation tests. Reliable. Considerable experience required</td></tr>
</table>

After A. Casagrande and R. E. Fadum

FIG. 2.8 Permeability and drainage characteristics of soils. (Table 6, p. 48, of *Soil Mechanics in Engineering Practice*, by K. Terzaghi and R. B. Peck, John Wiley and Sons, New York, 1948.)

Q:\CADD2\01FILES\95-00112030.00\246\GUADFIG023.DWG

Project No.
95-00112030.00

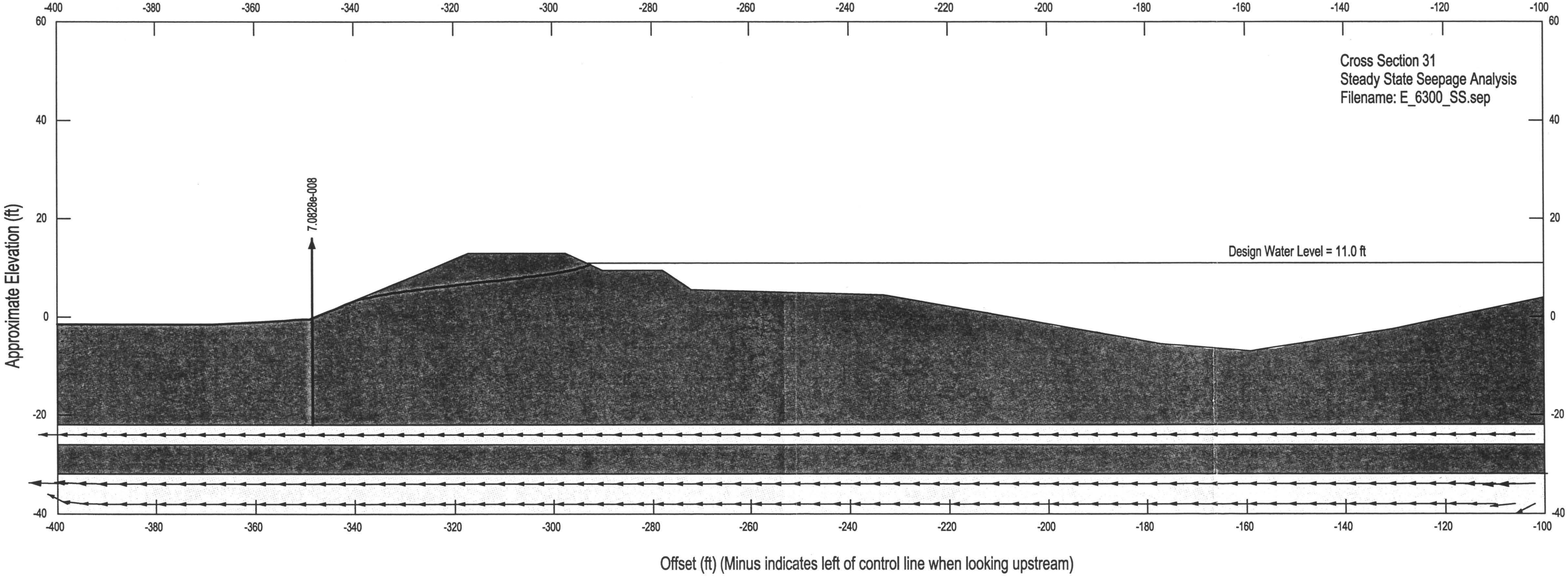
7/15/02

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara County, California

URS

## PERMEABILITY AND DRAINAGE CHARACTERISTICS OF SOILS

Figure
23


Cross Section 31
Steady State Seepage Analysis
Filename: E_6300_SS.sep
7.0828e-008
Design Water Level = 11.0 ft
Approximate Elevation (ft)
Offset (ft) (Minus indicates left of control line when looking upstream)
-400
-380
-360
-340
-320
-300
-280
-260
-240
-220
-200
-180
-160
-140
-120
-100
60
40
20
0
-20
-40

q:\cadd2\01files\95-00112030.00\246\ssseepage31.dwg

Project No.
95-00112030.00

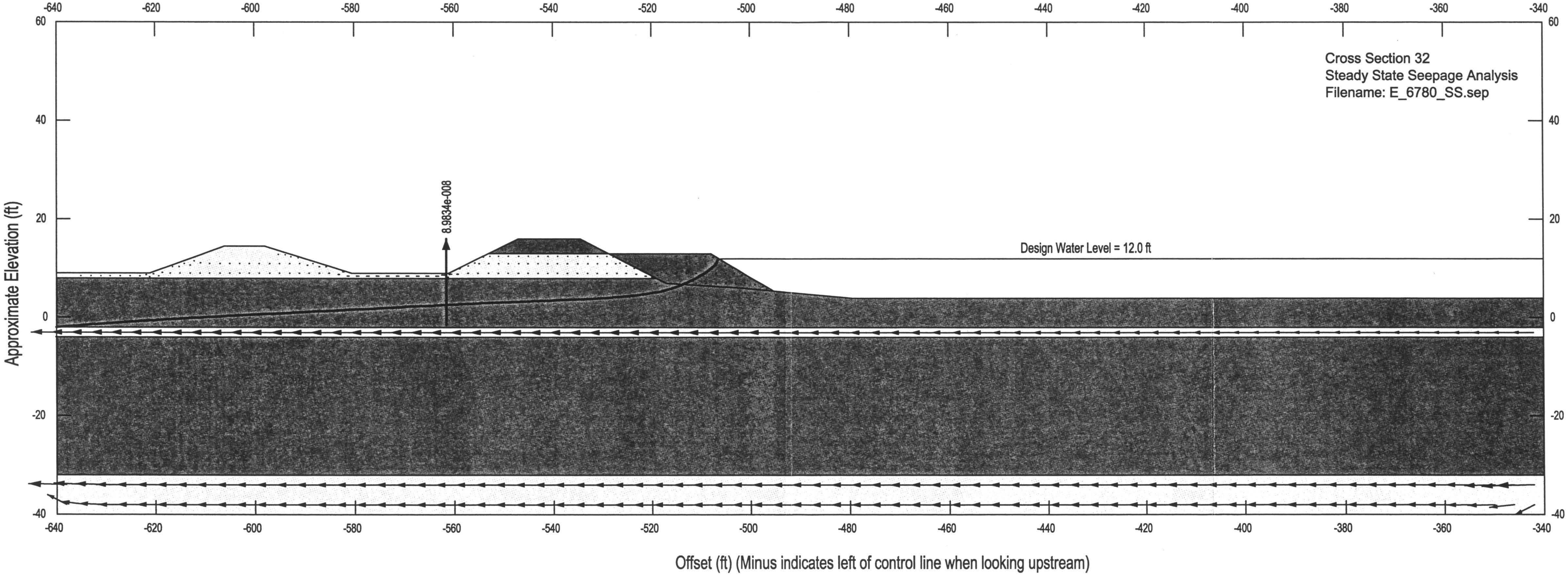
6/14/02

LOWER GUADALUPE RIVER
FLOOD CONTROL PROJECT
Santa Clara County, California

URS

**STEADY STATE SEEAGE ANALYSIS
CROSS SECTION 31**

Figure
24



q:\cadd2\01files\95-00112030.00\246\ssseepage32.dwg

| Project No.<br>95-00112030.00 | LOWER GUADALUPE RIVER<br>FLOOD CONTROL PROJECT<br>Santa Clara County, California | STEADY STATE SEEAGE ANALYSIS<br>CROSS SECTION 32 | Figure<br>25 |
|---|---|---|---|
| TEMP004 6/14/02 | URS | | |

# APPENDIX A

A total of seven (7) exploratory borings were performed at the subject site under the supervision of a professional from our firm. At most borings the rotary wash method was used as drilling mud was introduced at the ground surface. Some borings were initially advanced by the auger method; when a depth of about 4 meters (13 feet) was achieved or if groundwater was encountered at a shallower depth, drilling mud was introduced and the remainder of the hole was completed by the rotary wash method. Borings were drilled during the period of September 26 and 28, 2001, and January 15 and 17, 2002 to depths ranging from 10.2 to 24.8 meters (33.5 to 81.5 feet). Samples of the soil encountered in the exploratory borings were collected with a modified California sampler 50.8 mm (2-inch) inside diameter, 63.5 mm (2½ inch) outside diameter, California sampler 63.5 mm (2.5 inches) in diameter, 82.6 mm (3.25 inch) outside diameter or a standard split spoon sampler with 35 mm (1 3/8 inch) inside diameter, 51 mm (2 inch) outside diameter. All samplers were driven into the soil with a 63.6 kg (140 pound) hammer free-falling 762 mm (30 inches). Typically, the sampler was driven 457 mm (18 inches) into the soil, and the blow count was recorded for the final 305 mm (12 inches). When very dense or hard materials were encountered and the blow count exceeded 50 blows per 152 mm (6 inches), refusal criteria was met. The amount of penetration was then recorded along with the blow count. In addition Shelby tubes were pushed hydraulically to obtain cohesive samples; the hydraulic pressure was also recorded.

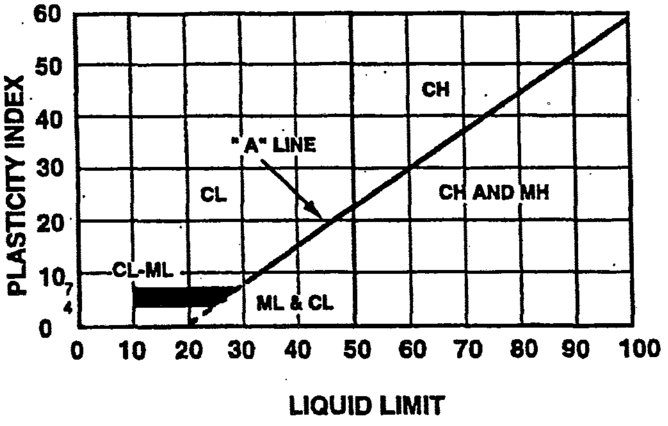
After driving, the sampler was withdrawn from the borehole, the retrieved soil was examined and classified, and the brass liners containing the samples were sealed to preserve the natural moisture. The samples were taken to our geotechnical laboratory for further examination and testing.

# SAMPLE CLASSIFICATION CHART

## UNIFIED SOIL CLASSIFICATION SCHEME

| MAJOR DIVISIONS | | SYMBOLS | TYPICAL NAMES |
|---|---|---|---|
| COARSE GRAINED SOIL (More than 1/2 of soil >no. 200 sieve size) | **GRAVELS** (More than 1/2 of coarse fraction > no. 4 sieve size) | GW | Well-graded gravels and gravel-sand mixtures, little or no fines |
| | | GP | Poorly graded gravel or gravel-sand mixtures, little or no fines |
| | | GM | Silty gravels, gravel-sand-silt mixtures |
| | | GC | Clayey gravels, gravel-sand-clay mixtures |
| | **SAND** (More than 1/2 of coarse fraction < no. 4 sieve size) | SW | Well-graded sands or gravelly sands, little or no fines |
| | | SP | Poorly-graded sands or gravelly sands, little or no fines |
| | | SM | Silty sands, sand-silt mixtures |
| | | SC | Clayey sands, sand-clay mixtures |
| FINE GRAINED SOIL (More than 1/2 of soil <no. 200 sieve size) | **SILTS & CLAYS** Liquid Limit < 50 | ML | Inorganic silts and very fine sands, rock flour, silty or clayey, fine sands or clayey silts with slight plasticity |
| | | CL | Inorganic clays of low to medium plasticity, gravelly clays. sandy clays, silty clays, lean clays |
| | | OL | Organic silts and organic silty clays of low plasticity |
| | **SILTS & CLAYS** Liquid Limit > 50 | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts |
| | | CH | Inorganic clays of high plasticity, fat clays |
| | | OH | Organic clays of medium to high plasticity, organic silts |
| HIGHLY ORGANIC SOILS | | Pt | Peat and other highly organic soils |

## PLASTICITY CHART



## GRAIN SIZE CLASSIFICATION

| CLASSIFICATION | RANGE OF GRAIN SIZES | |
|---|---|---|
| | U.S. Standard Sieve Size | Grain Size in Millimeters |
| BOULDERS | Above 12" | Above 305 |
| COBBLES | 12" to 3" | 305 to 76.2 |
| GRAVEL<br>coarse (c)<br>fine (f) | 3" to No. 4<br>3" to 3/4"<br>3/4" to No. 4 | 76.2 to 4.76<br>76.2 to 19.1<br>19.1 to 4.76 |
| SAND<br>coarse (c)<br>medium (m)<br>fine (f) | No. 4 to No. 200<br>No. 4 to No. 10<br>No. 10 to No. 40<br>No. 40 to No. 200 | 4.76 to 0.074<br>4.76 to 2.00<br>2.00 to 0.420<br>0.420 to 0.074 |
| SILT & CLAY | Below No. 200 | Below 0.074 |

## MOISTURE CONTENT

| | |
|---|---|
| DRY | -No sign of water and soil dry to touch |
| MOIST | -Signs of water and soil is relatively dry to touch |
| WET | -Signs of water and soil definitely wet to touch; granular soil exhibits some free water when densified |

## SOIL CONSISTENCY/RELATIVE DENSITY

| SILT, SAND AND GRAVEL | BLOWS/FT | SILT OR CLAY | UNCONFINED COMPRESSIVE STRENGTH (psf) | THUMB PENETRATION |
|---|---|---|---|---|
| Very loose | <4 | Very Soft | < 500 | Very easily - inches |
| Loose | 5-10 | Soft | 500 - 1000 | Easily - inches |
| Medium Dense | 11-30 | Medium (firm) | 1000 - 2000 | Moderate effort - inches |
| Dense | 31-50 | Stiff | 2000 - 4000 | Indented easily |
| Very Dense | >50 | Very Stiff | 4000 - 8000 | Indented by nail |
| | | Hard | > 8000 | Difficult by nail |

## CLASSIFICATION MODIFIERS

| | |
|---|---|
| TRACE | 0 - 12% |
| SOME | 12 - 30% |

± MODIFIERS

URS

Figure A-1

8/5/02 JGLOGPP 01030

**Project: LOWER GUADALUPE RIVER**
**Location: Santa Clara County, CA**

# Log of Boring LEGEND

Date Drilled:
Type of Boring: (as noted)
Hammer/drop: (as noted)

Remarks:
Surface Elevation: feet (approx.)

| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | Graphic Log | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | Arrow denotes bottom of fill layer<br>FILL ↑ | | | | |
| 5 | ▨ | | | | ← 2 inch inside diameter Modified California sample | | | | |
| | ▯ | | | | ← 2 inch outside diameter Standard Split Spoon sample (Standard Penetration Test) | | | | |
| 10 | | | | | | | | | |
| | ⊠ | | | | ← 3 inch outside diameter Shelby tube sample | | | | |
| 15 | | | | | | | | | |
| | ⊠ | 350 psi | | | ← Hydraulic Pressure required to push Shelby tube sampler | | | | |
| 20 | ■ | 29 | | | ← Blow count with 140-lb hammer falling 30 inches for 12 inches of penetration | | | | |
| 25 | ■ | 50/ 5" | | | ← Blow count with 140-lb hammer falling 30 inches for 5 inches of penetration | | | | |
| | | | | | ▽ Groundwater level at time of drilling | | | | |
| 30 | | | | | | | | | |
| | | | | | ▼ Groundwater at a time after drilling (as specified) | | | | |
| 35 | | | | | **KEY TO LABORATORY TESTS** | | | | |
| | | | 3.0 | | PP = Pocket Penetrometer reading in tons per square foot (tsf) | | | | PP = 3.0tsf |
| | | | | | LL = Liquid Limit (%)<br>PI = Plasticity Index (%)<br>NOTE: PI = LL - (Plastic Limit [%]) | | | | LL = 42<br>PI = 21 |
| 40 | | | | | +#4 = Percentage of material retained on #4 sieve<br>-#200 = Percentage of material passing #200 sieve | | | | +#4 = 13%<br>-#200 = 10% |
| 45 | | | | | | | | | |

**Project: LOWER GUADALUPE RIVER**
**Location: Santa Clara County, California**

# Log of Boring EB-01

Date Drilled: 5/5/98
Type of Boring: 6in Auger; 4-7/8in Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Augered to 17-1/2 feet, sampled, then began rotary wash
Surface Elevation: 6.9 feet (approx.)

| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | Graphic Log | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | **Poorly graded GRAVEL (GP-GC) with clay - FILL**<br>Cobbles to 4 inches | | | | |
| | 1 | 28 | >4.5 | | **Lean to fat CLAY (CL/CH) with sand - FILL**<br>Hard, moist, dark greenish gray, fine to coarse sand, trace gravel | | | | |
| | 2 | 37 | >4.5 | | | 13 | 121 | 6570 | |
| 5 | 3 | 37 | >4.5 | | Increase in gravel | | | | |
| | 4 | 26 | | | Alternating sand and gravel fill, brick pieces | | | | |
| | 5 | 9 | 0.7 | | Dark brown | | | | |
| 10 | 6 | 200<br>450 psi | 3.0 | | | | | | |
| | 7 | 14 | 0.5 | | Increase in sand and gravel | | | | |
| | 8 | 13 | 1.5 | | FILL | | | | |
| 15 | 9 | 25 | 3.0 | | **Well-graded SAND (SW-SC) with clay and gravel**<br>Very dense, wet, dark greenish gray, fine to coarse sand | | | | |
| | 10 | 57 | | | | | | | |
| | 11 | 60 | | | | | | | |
| 20 | | | | | Decrease in gravel | | | | |
| | 12 | 14 | 1.0 | | **Fat CLAY (CH)**<br>Medium, moist, dark brownish gray, very high plasticity | | | | |
| | 13 | 200<br>300 psi | 1.2 | | | 25 | 101 | | |
| 25 | 14 | 240<br>500 psi | 0.5 | | | | | | |
| 30 | | | | | | | | | |
| | 15 | 30 | | | **Lean CLAY (CL)**<br>Stiff, moist, brown, some sand | 18 | 114 | 4260 | |
| 35 | | | | | | | | | |
| | 16 | 32 | | | **Poorly graded SAND (SP)**<br>Dense, wet, dark brown, fine sand | | | | |
| 40 | 17 | 34 | | | | | | | |
| | 18 | 32 | 2.2 | | **Lean to fat CLAY (CL/CH)**<br>Very stiff, moist, brown mottled light gray, medium to high plasticity | | | | |
| 45 | | | | | BOTTOM OF BORING AT 44-1/2 FEET | | | | |
| 50 | | | | | | | | | |

8/5/02 JGLOGPP 01030

**Project: LOWER GUADALUPE RIVER**
**Location: Santa Clara County, CA**

# Log of Boring EB-27

Date Drilled: 9/28/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Santa Clara
Surface Elevation: 13 feet (approx.)

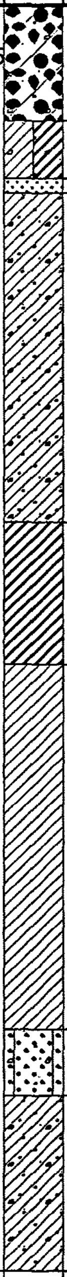
| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | Graphic Log | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | **Silty GRAVEL (GM) with sand - FILL**<br>Medium dense, dry, brownish gray, gravel to 2-1/2 inches | | | | |
| 5 | 1 | 20 | | | **Gravelly lean CLAY (CL)**<br>Very stiff, moist, brownish gray, gravel to 1 inch | | | | Began rotary wash at 5' |
| 10 | 2 | 10 | 0.5 | | Soft to medium, gray to reddish brown | 26 | | | |
| | 3 | 0<br>50<br>psi | | | FILL | | | | |
| 15 | 4 | 5 | | | **Poorly graded SAND (SP)**<br>Loose, wet, light brownish gray, fine to medium sand | | | | No recovery |
| | 5 | 60<br>psi | | | **Lean CLAY (CL)**<br>Medium, moist to wet, light brownish gray | | | | Consol |
| 20 | | | | | | | | | |
| 25 | 6 | 6 | | | **Fat CLAY (CH)**<br>Soft, moist, dark reddish brown and gray mottled, trace sand and decomposed organics, slight organic odor | 48 | 70 | 570 | LL=55<br>PI=27 |
| 30 | 7 | 70<br>125<br>psi | | | Occasional sand layers | | | | |
| | | | | | Trace fine gravel | | | | |
| 35 | 8 | 3 | 0.5 | | Wet, dark gray, wood fragments, trace sand | 56 | 63 | 940 | LL=76<br>PI=42 |
| 40 | 9 | 100<br>psi | | | | | | | Consol |
| 45 | | | | | BOTTOM OF BORING AT 44 FEET | | | | |
| 50 | | | | | | | | | |

**Project: LOWER GUADALUPE RIVER**
**Location: Santa Clara County, CA**

# Log of Boring EB-28

Date Drilled: 9/28/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Santa Clara
Surface Elevation: 12.5 feet (approx.)



| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|
| | 1 | 32 | >4.5 | **Clayey GRAVEL (GC) with sand - FILL**<br>Medium dense, moist, brown and gray mottled, gravel to 1-1/2 inch | | | | |
| 5 | 2 | 23 | | **Lean to fat CLAY (CL/CH) with sand - FILL**<br>Medium, moist, brown<br>FILL | | | | Began rotary wash at 5' |
| | | | | **Poorly graded SAND (SP)**<br>Medium dense, moist to wet, brown | | | | |
| 10 | 3 | 80<br>125<br>psi | | **Sandy lean CLAY (CL)**<br>Medium, moist, brownish gray | | | | |
| 15 | 4 | 8 | | Frequent sand lenses | 21 | 99 | 1500 | |
| | | | | **Fat CLAY (CH)**<br>Stiff, moist, gray brown | | | | |
| 20 | 5 | 20 | | | 27 | 98 | 2970 | LL=69<br>PI=47 |
| | | | | **Lean CLAY (CL)**<br>Stiff, moist, grayish brown and light brown, trace clay nodules | | | | |
| 25 | 6 | 125<br>180<br>250<br>psi | | | | | | No recovery |
| 30 | 7 | 21 | | Brown mottled | 27 | 95 | 2240 | |
| 35 | 8 | 41 | | **Silty SAND (SM)**<br>Dense, wet, brown, fine sand | | | | |
| | | | | **Sandy lean CLAY (CL)**<br>Stiff, moist, gray | | | | |
| 40 | 9 | 80<br>psi | | | | | | |
| 45 | | | | BOTTOM OF BORING AT 44 FEET | | | | |
| 50 | | | | | | | | |

Project: 95-00112030.00

URS

Sheet 1 of 1

Fig. A-5

8/5/02 JGLOGPP 01030

**Project: LOWER GUADALUPE RIVER**
**Location: Santa Clara County, CA**

# Log of Boring CB-14

Date Drilled: 9/26/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Gold Street Bridge
Surface Elevation: 6.2 feet (approx.)

| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | Graphic Log | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | **Clayey SAND (SC) with gravel - FILL**<br>Dense, damp, gray | | | | |
| | 1 | 49 | | | FILL | | | | |
| | 2 | 10 | 2.0 | | **Lean CLAY (CL) with sand**<br>Stiff, moist, brown and gray brown mottled | | | | |
| 5 | 3 | 9 | | | | 22 | 82 | | |
| | 4 | 4 | | | **Fat CLAY (CH)**<br>Soft, moist, gray, organic material, organic odor, well graded sand, fine gravel | 73 | 54 | 590 | |
| | 5 | 41 | | | | 63 | 60 | 630 | |
| | | | | | **Silty GRAVEL (GM) with sand**<br>Dense, wet, brown to gray | | | | |
| 10 | 6 | 41 | | | | 12 | 110 | | Began rotary wash at 10' |
| | | | | | **Clayey GRAVEL (GC) with sand**<br>Dense, wet, gray brown, well graded sand, fine gravel | | | | |
| | 7 | 26 | | | | | | | Sleeve #1 full of crushed shells |
| | | | | | Light brown Silty Sand (SM) from 12 to 13 feet | | | | |
| | | | | | **Lean CLAY (CL) with gravel**<br>Hard, moist, light brown | | | | |
| 15 | | | | | **Fat CLAY (CH)**<br>Soft to medium, moist, dark gray, organic odor, some organics | | | | |
| | 8 | 5 | 0.5 | | | | | | |
| 20 | | | | | | | | | |
| | 9 | 4 | | | | 42 | 79 | 580 | |
| 25 | | | | | **Silty SAND (SM)**<br>Loose, moist, gray, fine sand | | | | |
| | 10 | 5 | | | | | | | |
| 30 | | | | | | | | | |
| | 11 | 7 | | | Increase in plasticity of fines | 23 | 106 | | +#4=0<br>-#200=40% |
| | | | | | BOTTOM OF BORING AT 33-1/2 FEET | | | | |
| 35 | | | | | | | | | |
| 40 | | | | | | | | | |
| 45 | | | | | | | | | |
| 50 | | | | | | | | | |

**Project: LOWER GUADALUPE RIVER**
**Location: Santa Clara County, CA**

# Log of Boring CB-15

Date Drilled: 9/26/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Gold Street Bridge Near Yacht Club
Surface Elevation: 4.5 feet (approx.)

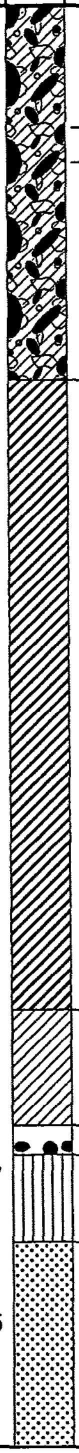
| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|
| | | | | **Lean CLAY (CL) with gravel - FILL**<br>Soft, moist, brown, some roots | | | | |
| | 1 | 7 | 1.0 | FILL | | | | |
| | | | | **Fat CLAY (CH)**<br>Soft, moist, gray, trace sand | | | | |
| | 2 | 3 | | | 43 | 74 | 650 | |
| 5 | 3 | 4 | | ∇ | 38 | 79 | 380 | |
| | 4 | 2 | | | | | | Began rotary wash at 6' |
| | 5 | 2 | | | 63 | | | |
| 10 | 6 | 2 | <0.5 | | | | | |
| | | | | **Clayey SAND to Sandy fat CLAY (CH/SC)**<br>Soft to medium, wet, gray, slight organic odor, thin fine to medium sand lenses | | | | |
| | 7 | 2 | | | 51 | 70 | 330 | |
| 15 | | | | | | | | |
| | 8 | 5 | | | 42 | 76 | 610 | |
| 20 | | | | | | | | |
| | 9 | 10 | | | 28<br>41 | 90<br>76 | 360<br>1040 | |
| 25 | | | | **Fat CLAY (CH)**<br>Soft to medium, wet, grayish brown and black, trace fine to medium sand, slight organic odor | | | | |
| | 10 | 6 | | | 64 | 60 | 810 | |
| 30 | | | | | | | | |
| | 11 | 7 | | With fine sand | 31 | 90 | 1330 | |
| 35 | | | | BOTTOM OF BORING AT 33-1/2 FEET | | | | |
| 40 | | | | | | | | |
| 45 | | | | | | | | |
| 50 | | | | | | | | |

**Project: LOWER GUADALUPE RIVER**
**Location: Santa Clara County, CA**

# Log of Boring CB-16

Date Drilled: 1/15/02
Type of Boring: 4-7/8 inch Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Cargill Salt Property
Surface Elevation: 10.8 feet (approx.)



| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|
| | | | | **MISCELLANEOUS DEBRIS Lean CLAY (CL) with sand and gravel; Concrete Chunks and Slabs - FILL**<br>Very stiff, moist, brown to dark gray | | | | |
| 5 | 1 | 16 | 2.0 | Trace roots | 22 | | | |
| 10 | 2 | 25 | 0.5 | Soft | 21 | | | |
| | | | | FILL | | | | |
| | | | | **Fat CLAY (CH)**<br>Soft, moist, olive gray to dark gray, trace sand and rootlets | | | | |
| 15 | 3 | 5 | | | 70 | 58 | 920 | Dry to 14'; began rotary wash |
| 20 | 4 | 50 psi | | | 76 | 54 | 640 | |
| 25 | 5 | 5 | 0.5 | | 78 | 53 | 610 | |
| 30 | 6 | 5 | 0.5 | | 67 | 59 | 1130 | |
| 35 | 7 | 75 psi | | **Lean CLAY (CL)**<br>Wet, dark olive gray | 25 | 101 | 1100 | |
| | | | | Gravel layer encountered - 39-1/2 to 40 feet | | | | |
| 40 | 8 | 12 | 0.7 | **SILT (ML)**<br>Medium, wet, olive gray, trace fine sand | 26 | 100 | 1960 | |
| | | | | **Poorly graded SAND (SP)**<br>Medium dense, wet, brown to olive gray, fine to medium sand, trace silt | | | | |
| 45 | 9 | 28 | 0.5 | | | | | |
| | | | | Olive gray, trace clay | | | | |
| 50 | | | | | | | | |

**Project: LOWER GUADALUPE RIVER**

# Log of Boring CB-16 Continued

| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | Graphic Log | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|---|
| | 10 | 16 | | | **Silty SAND (SM)**<br>Medium dense, wet, brown to olive green | 26 | | | +#4=0<br>-#200=48% |
| 55 | 11 | 34 | 0.5 | | Dense, dark gray | | | | |
| 60 | 12 | 32 | | | | | | | |
| | | | | | **Lean CLAY (CL)**<br>Stiff, moist, greenish gray to brownish yellow mottled, trace fine sand | | | | |
| 65 | 13 | 48 | 2.0 | | **Poorly graded SAND (SP)**<br>Dense, wet, brown to brownish yellow, fine sand, trace clay | 15 | 119 | 1400 | |
| 70 | 14 | 54 | 0.5 | | Medium sand | | | | |
| 75 | 15 | 43 | | | **Well-graded SAND (SW)**<br>Dense, wet, brown to dark gray, fine to coarse sand, trace silt and fine gravel | | | | |
| 80 | 16 | 45 | | | Trace fine gravel | | | | |
| | | | | | BOTTOM OF BORING AT 81-1/2 FEET | | | | |
| 85 | | | | | | | | | |
| 90 | | | | | | | | | |
| 95 | | | | | | | | | |
| 100 | | | | | | | | | |
| 105 | | | | | | | | | |

**Project: LOWER GUADALUPE RIVER**
**Location: Santa Clara County, CA**

# Log of Boring CB-17

Date Drilled: 1/16/02
Type of Boring: 4-7/8 inch Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Cargill Salt Property
Surface Elevation: 10.8 feet (approx.)

| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|
| | | | | **MISCELLANEOUS DEBRIS - FILL**<br>Concrete, wood and asphalt | | | | |
| 5 | | | | | | | | |
| 10 | | | | Brick | | | | |
| | | | | Took 3-1/2 hours to drill to 13 feet | | | | Dry to 13'; began rotary wash |
| | | | | FILL | | | | |
| 15 | 1 | 3 | | **Fat CLAY (CH)**<br>Soft, moist, dark gray to brown mottled, trace rootlets | 76 | 55 | 640 | |
| 20 | 2 | 50 psi | | | 77 | 54 | 740 | Consol |
| 25 | 3 | 2 | | Trace silt | 56 | 67 | 570 | |
| 30 | 4 | 50 psi | | Wet, greenish gray to olive gray | 55 | 67 | 1200 | Consol |
| 35 | 5 | 3 | | With seams of fine to medium sand | 55 | 68 | 870 | |
| 40 | 6 | 50 psi<br>100 psi | | | | | | |
| | | | | Sand layer encountered - 42 to 44 feet | | | | |
| 45 | 7 | 5 | 0.5 | | 55 | 68 | 920 | |
| | | | | **Lean CLAY (CL) with gravel**<br>Stiff to very stiff, moist to wet, greenish gray | | | | |
| 50 | | | | | | | | |

**Project: LOWER GUADALUPE RIVER**

## Log of Boring CB-17 Continued

| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | Graphic Log | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|---|
|  | 8 | 41 |  |  | to olive gray |  |  |  |  |
|  |  |  |  |  | **Poorly graded SAND (SP)**<br>Dense, wet, greenish gray to olive gray | 20 | 109 |  |  |
| 55 | 9 | 33 |  |  |  |  |  |  |  |
| 60 | 10 | 38 |  |  | **Poorly graded GRAVEL (GP) with sand**<br>Dense, wet, greenish gray to olive gray |  |  |  |  |
|  |  |  |  |  | **Well-graded SAND (SW) with gravel**<br>Dense, wet, greenish gray to dark gray, trace silt |  |  |  |  |
| 65 | 11 | 57 | 1.5 |  | **Poorly graded SAND (SP-SM) with silt and gravel**<br>Dense to very dense, wet, brown to brownish yellow, medium to coarse sand, trace medium to fine gravel |  |  |  |  |
| 70 | 12 | 29 |  |  |  | 10 |  |  | +#4=23%<br>-#200=8% |
| 75 | 13 | 43 |  |  | **Well-graded GRAVEL (GW) with sand**<br>Dense, wet, greenish gray to gray, trace fine sand |  |  |  |  |
|  |  |  |  |  | **Well-graded SAND (SW)**<br>Medium dense, wet, dark gray, medium to coarse |  |  |  |  |
| 80 | 14 | 28 |  |  |  |  |  |  |  |
|  |  |  |  |  | BOTTOM OF BORING AT 81-1/2 FEET |  |  |  |  |
| 85 |  |  |  |  |  |  |  |  |  |
| 90 |  |  |  |  |  |  |  |  |  |
| 95 |  |  |  |  |  |  |  |  |  |
| 100 |  |  |  |  |  |  |  |  |  |
| 105 |  |  |  |  |  |  |  |  |  |

**Project: LOWER GUADALUPE RIVER**
**Location: Santa Clara County, CA**

# Log of Boring CB-18

Date Drilled: 1/17/02
Type of Boring: 4-7/8 inch Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Cargill Salt Property
Surface Elevation: 10.5 feet (approx.)

| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | Graphic Log | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | **MISCELLANEOUS DEBRIS - FILL**<br>Wood, carpeting, concrete chunks and slabs | | | | |
| 5 | | | | | | | | | |
| 10 | | 15 | | | No sample, wood debris, carpeting | | | | No recovery |
| | 1 | 2 | | | Trace silt, carpet fibers and roots at 12 feet<br>FILL | 45 | 75 | 680 | |
| 15 | | | | | **Fat CLAY (CH)**<br>Very soft to soft, wet, greenish gray to olive gray | | | | Began rotary wash at 13' |
| | 2 | 50 psi | | | | 48 | 73 | 500 | |
| 20 | 3 | 2 | <0.5 | | With roots | | | | |
| 25 | 4 | 2 | | | Trace roots | 76 | 56 | 640 | |
| 30 | 5 | 50 psi | <0.5 | | | 65 | 60 | 840 | |
| 35 | 6 | 4 | <0.5 | | **Elastic SILT (MH)**<br>Loose, wet, brown, trace fine sand and gravel | | | | |
| 40 | 7 | 9 | <0.5 | | | 27 | | | |
| 45 | 8 | 125 psi | 1.2 | | Loose to medium dense | | | | |
| | | | | | **Poorly graded SAND (SP)**<br>Dense, wet, brown to dark brown, fine to | | | | |
| 50 | | | | | | | | | |

**Project: LOWER GUADALUPE RIVER**

# Log of Boring CB-18 Continued

| Depth, feet | Samples | Blows/ft | Pocket Pen. (tsf) | Graphic Log | MATERIAL DESCRIPTION | Moisture Content, % | Dry Density, pcf | Unconfined Compressive Strength, psf | Other Tests/ Remarks |
|---|---|---|---|---|---|---|---|---|---|
| | 9 | 37 | | | medium | | | | |
| 55 | 10 | 18 | | | Medium dense | | | | |
| 60 | 11 | 50 | | | **Poorly graded SAND (SP-SM) with silt and gravel**<br>Dense to very dense, wet, brown to dark gray | 11 | 126 | | +#4=18%<br>-#200=9% |
| | | | | | **Lean CLAY (CL)**<br>Medium to stiff, wet, olive gray to brownish yellow mottled | | | | |
| 65 | 12 | 25 | 2.0<br>1.7 | | **Clayey SAND (SC)**<br>Medium dense, wet, olive gray to brownish yellow mottled, fine sand | | | | |
| 70 | 13 | 17 | | | | | | | No recovery |
| 75 | 14 | 50 | 1.0 | | **Poorly graded SAND (SP)**<br>Dense to very dense, wet, greenish gray to dark gray, fine to medium sand | | | | |
| 80 | 15 | 27 | | | **Well-graded SAND (SW)**<br>Medium dense, wet, olive gray to dark gray, fine to coarse sand | | | | |
| | | | | | BOTTOM OF BORING AT 81-1/2 FEET | | | | |
| 85 | | | | | | | | | |
| 90 | | | | | | | | | |
| 95 | | | | | | | | | |
| 100 | | | | | | | | | |
| 105 | | | | | | | | | |

# APPENDIX B

A laboratory-testing program was implemented to obtain engineering properties from selected soil samples during the field exploration. The following tests were performed:

1. Moisture Content And Unit Weight: In-situ moisture content and in-situ dry unit weight was determined on selected soil samples in accordance with ASTM D2216 and D2937.

2. Unconfined Compressive Strength: The unconfined compressive strength was measured on samples retrieved from the sampler brass liners in accordance with ASTM D2166.

3. Atterberg Limits: Liquid limit (LL) and plastic limit (PL) were determined on selected samples using the procedure given in ASTM D4318. The plasticity index (PI) was computed from these values using the expression PI = LL – PL.

4. Gradation Analyses: Gradation (Sieve) analyses were performed on selected samples to determine the total percentage of gravel (retained by the No. 4 standard sieve) and fines (passing the No. 200 sieve or finer than 74 μm). The tests were performed in accordance with the standard procedure specified by ASTM D422. Wash sieves were performed on selected samples to determine the percentage of fines. Percentages of gravel and fines are also shown in Boring Logs.

5. Consolidation Tests: Consolidation tests were performed on samples obtained from Shelby tubes at various depths. The tests were performed in accordance with the procedure specified by ASTM D 2435.

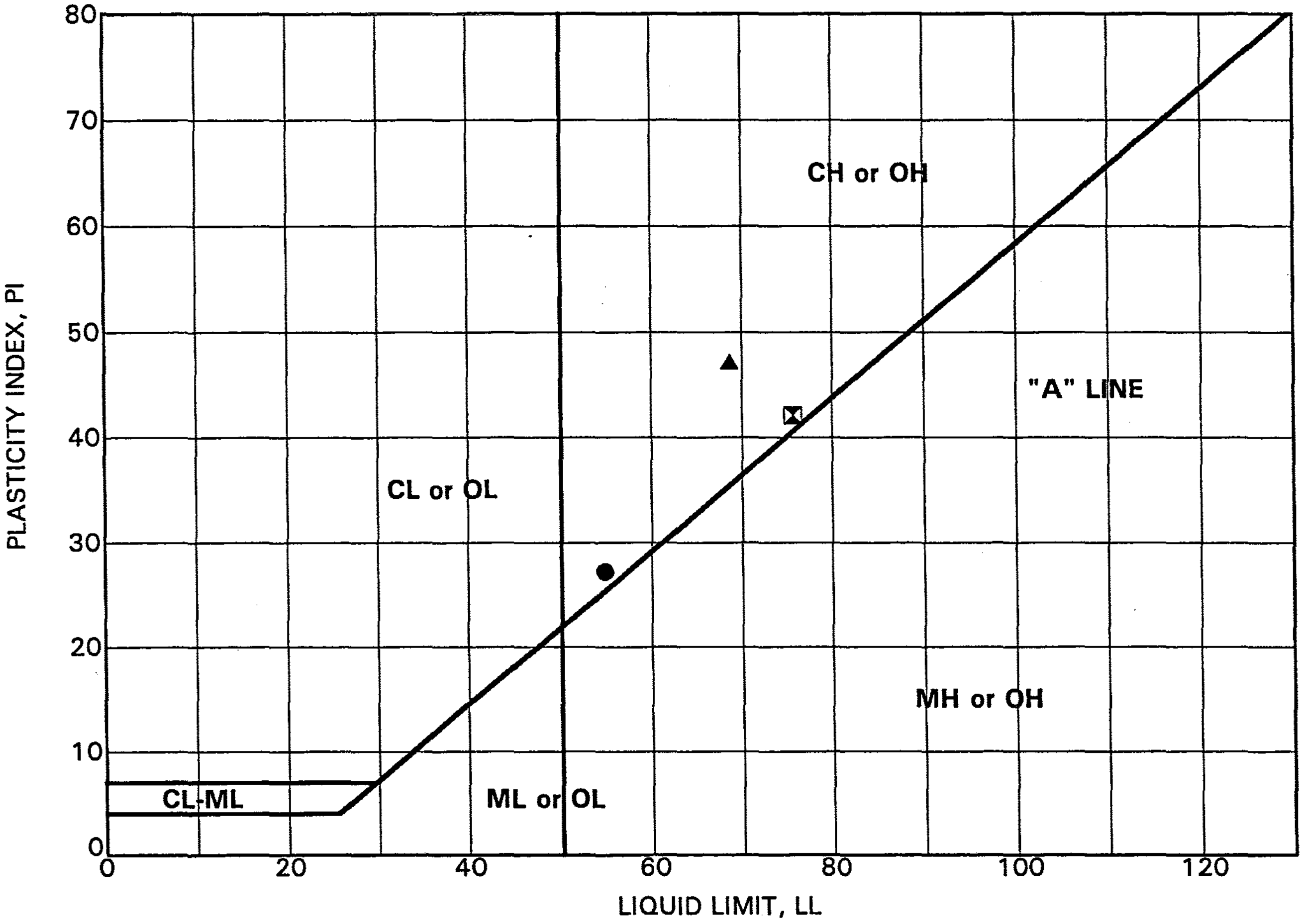
Laboratory test results are shown in figures in this Appendix and in the Boring Logs (Appendix A).

Section 1 Atterberg Limits

Section 2 Grain Size Distribution

Section 3 Consolidation

# SECTION 1



| Boring Number | Sample Number | Depth (feet) | Test Symbol | Moisture Content (%) | LL | PL | PI | Description |
|---|---|---|---|---|---|---|---|---|
| EB-27 | 6 | 26 | ● | 48 | 55 | 28 | 27 | Fat CLAY (CH) |
| EB-27 | 8 | 36 | ☒ | 56 | 76 | 34 | 42 | Fat CLAY (CH) with sand |
| EB-28 | 5 | 21 | ▲ | 27 | 69 | 21 | 48 | Fat CLAY (CH) with sand |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |
|  |  |  |  |  |  |  |  |  |

**Project: LOWER GUADALUPE RIVER**
**Project Number: 95-00112030.00**

## PLASTICITY CHART

**Figure B-1**

URS

# SECTION 2

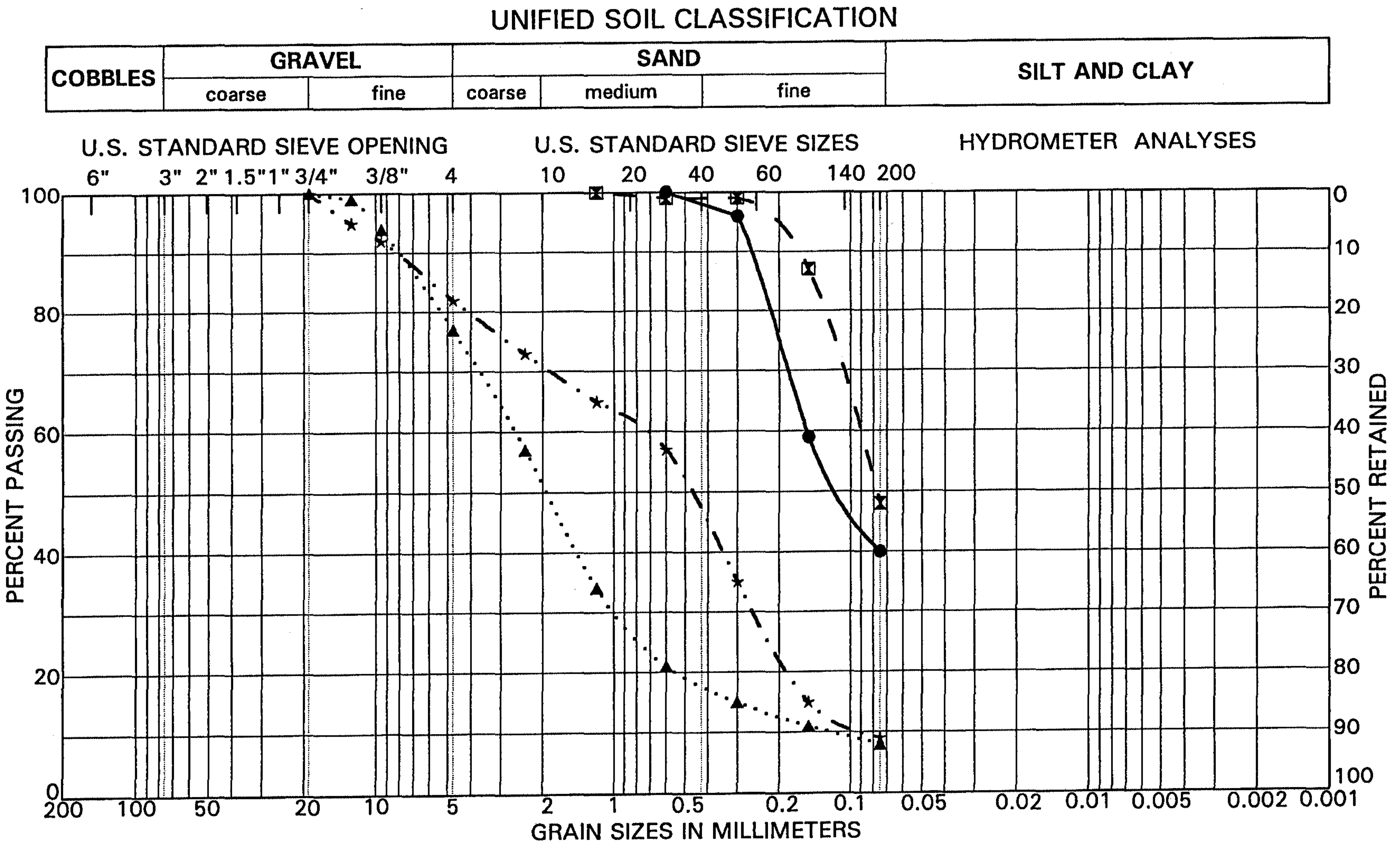
Project : LOWER GUADALUPE RIVER
Project No. 95-00112030.00

8/5/02 JSIEVE 01030

# GRAIN SIZE DISTRIBUTION CURVES

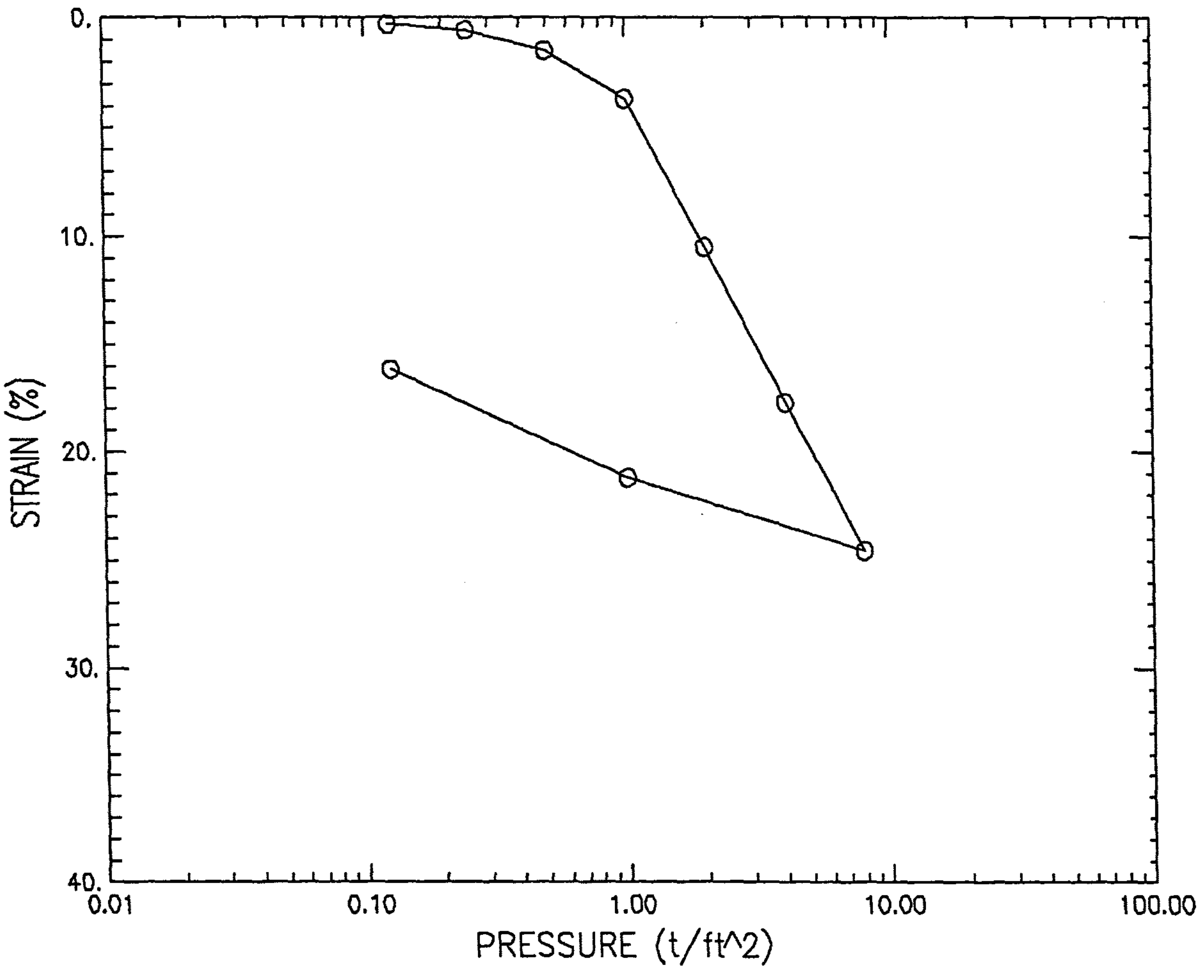
Fig. B-2

URS



| Boring Number | Sample Number | Depth (feet) | Symbol | LL | PI | Classification |
|---|---|---|---|---|---|---|
| CB-14 | 11 | 33 | ● | | | Silty SAND (SM) |
| CB-16 | 10 | 51 | ☒ | | | Silty SAND (SM/ML) |
| CB-17 | 12 | 71 | ▲ | | | Well-graded SAND (SW-SM) with silt and gravel |
| CB-18 | 11 | 61 | ★ | | | Poorly graded SAND (SP-SM) with silt and gravel |

# SECTION 3

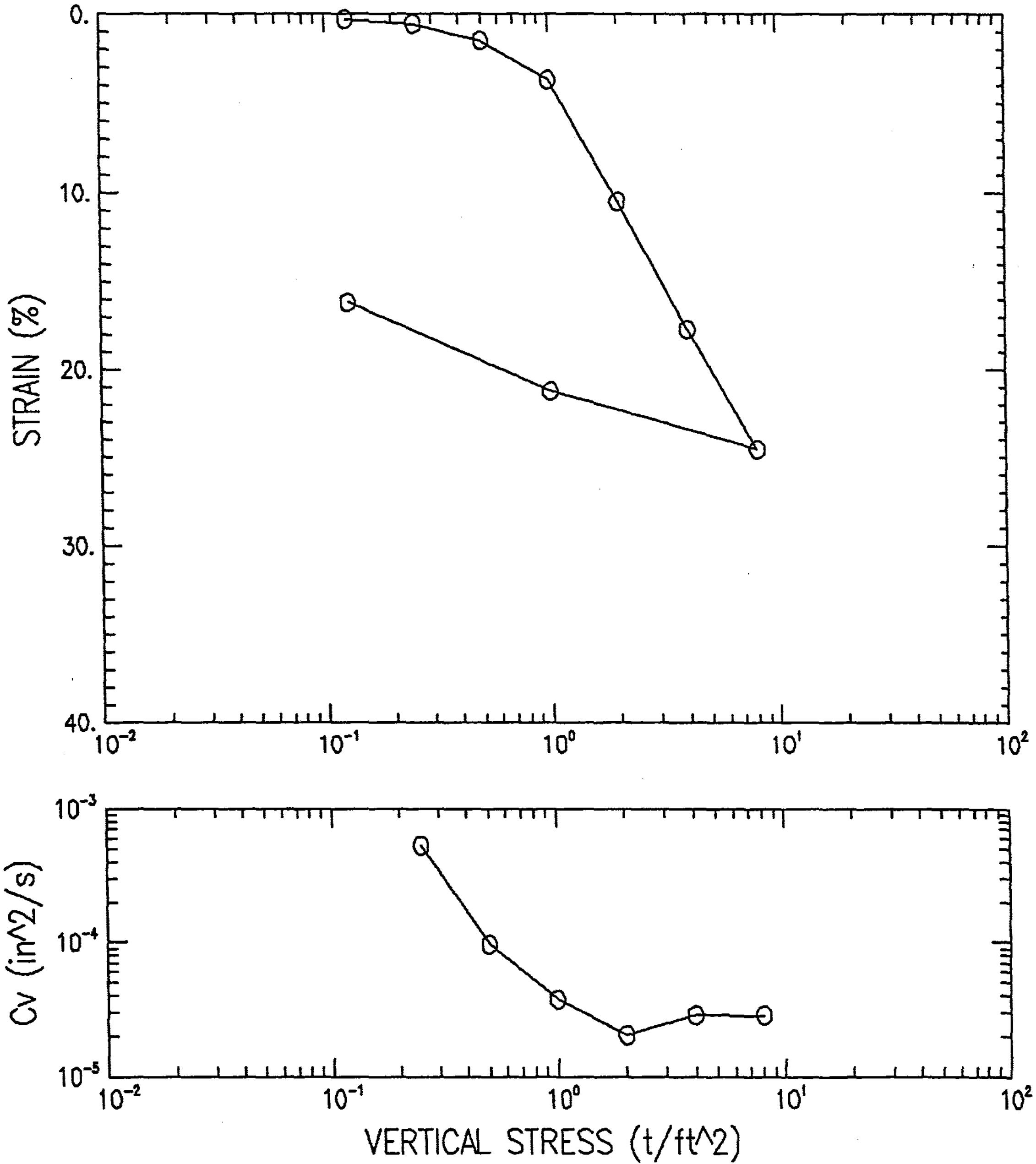


| | | | BEFORE TEST | AFTER TEST |
|---|---|---|---|---|
| OVERBURDEN PRESSURE (t/ft^2) | | WATER CONTENT (%) | 50.603 | 36.659 |
| PRECONSOL. PRESSURE (t/ft^2) | | DRY DENSITY (lb/ft^3) | 68.981 | 82.273 |
| COMPRESSION INDEX | | SATURATION (%) | 98.672 | 99.987 |
| TYPE SPECIMEN | | VOID RATIO | 1.308 | 0.935 |
| DIA. (In) 2.430 | HT. (In) 0.957 | BACK PRESSURE (t/ft^2) | | |

| | | | | |
|---|---|---|---|---|
| CLASSIFICATION Gray silty Clay with blk carbon & organic peat | | | | |
| LL 0.0 | PL 0.0 | PI 0.0 | PROJECT L. Guadalupe River Flood | |
| GS 2.550 | $D_{10}$ | | Data File: EB27-5.CON | |
| REMARKS | | | BORING NO. EB-27 | SAMPLE NO. 5 |
| | | | DEPTH 18-20 feet | DATE 10/24/01 |

Woodward Clyde Consultants
CONSOLIDATION TEST REPORT

# CONSOLIDATION TEST
# SUMMARY REPORT



Woodward Clyde Consultants
Project Name : L. Guadalupe River Flood
Project No : 9500112030 Boring No : EB-27 Sample No : 5
Test Date : 10/24/01 Test No : EB27-5 Depth : 18-20 feet
Description : Gray silty Clay with blk carbon & organic peat

Fri Nov 02 16:44:49 2001

# CONSOLIDATION TEST DATA

Project : L. Guadalupe River Flood
Location : San Jose, CA
Project No. : 9500112030
Test No. : EB27-5
Boring No. : EB-27
Test Date : 10/24/01
Tested by : C. Wason
Sample No. : 5
Sample Type : Shelby
Depth : 18-20 feet
Checked by : S. Capps
Soil Description : Gray silty Clay with blk carbon & organic peat
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

| | APPLIED PRESSURE (t/ft^2) | FINAL DISPLACEMENT (in) | VOID RATIO | STRAIN AT END (%) | FITTING TIME (min) | | COEFFICIENT OF CONSOLIDATION (in^2/s) | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | SQ.RT. | LOG | SQ.RT. | LOG | AVE |
| 1) | 0.13 | 0.003 | 1.301 | 0.30 | 0.0 | 0.0 | 0.00E+000 | 0.00E+000 | 0.00E+000 |
| 2) | 0.25 | 0.005 | 1.295 | 0.55 | 1.4 | 0.0 | 5.28E-004 | 0.00E+000 | 0.00E+000 |
| 3) | 0.50 | 0.014 | 1.274 | 1.48 | 7.7 | 0.0 | 9.58E-005 | 0.00E+000 | 0.00E+000 |
| 4) | 1.00 | 0.035 | 1.223 | 3.67 | 19.1 | 0.0 | 3.73E-005 | 0.00E+000 | 0.00E+000 |
| 5) | 2.00 | 0.100 | 1.066 | 10.48 | 31.9 | 0.0 | 2.04E-005 | 0.00E+000 | 0.00E+000 |
| 6) | 4.00 | 0.169 | 0.900 | 17.69 | 19.4 | 0.0 | 2.87E-005 | 0.00E+000 | 0.00E+000 |
| 7) | 8.00 | 0.235 | 0.741 | 24.55 | 16.4 | 0.0 | 2.85E-005 | 0.00E+000 | 0.00E+000 |
| 8) | 1.00 | 0.203 | 0.818 | 21.21 | 0.0 | 0.0 | 0.00E+000 | 0.00E+000 | 0.00E+000 |
| 9) | 0.13 | 0.155 | 0.935 | 16.15 | 0.0 | 0.0 | 0.00E+000 | 0.00E+000 | 0.00E+000 |

Fri Nov 02 16:44:49 2001

# CONSOLIDATION TEST DATA

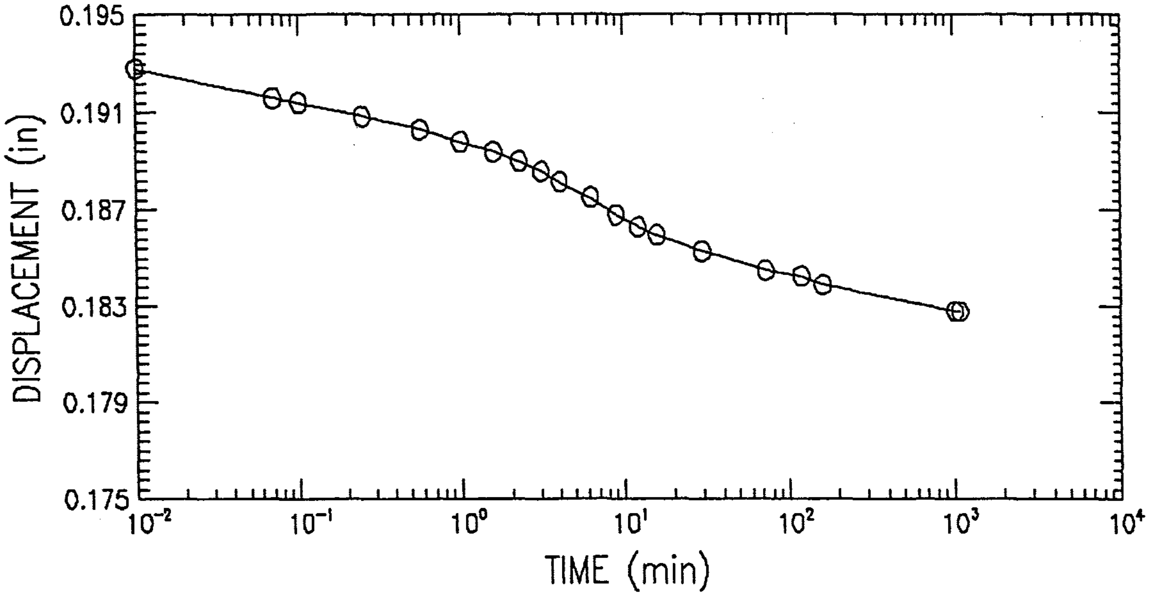
Project : L. Guadalupe River Flood
Location : San Jose, CA
Project No. : 9500112030
Test No. : EB27-5
Boring No. : EB-27
Test Date : 10/24/01
Tested by : C. Wason
Sample No. : 5
Sample Type : Shelby
Depth : 18-20 feet
Checked by : S. Capps
Soil Description : Gray silty Clay with blk carbon & organic peat
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

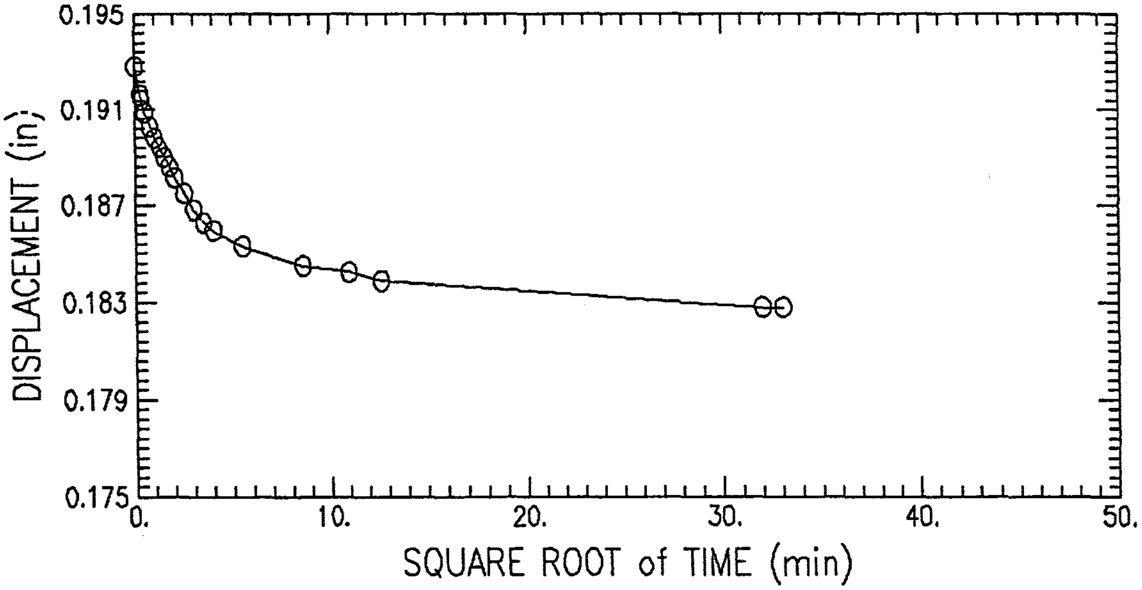
Liquid Limit : 0
Plastic Limit : 0
Plasticity Index : 0
Specific Gravity : 2.55
Initial Height : 0.96
Sample Diameter : 2.43 (in)
Initial Void Ratio : 1.3073
Weight of Piston and top cap : 0.00 (gm)

| | TRIMMINGS | SPECIMEN + RING BEFORE | SPECIMEN + RING AFTER | AFTER |
|---|---|---|---|---|
| CONTAINER NO. | | RING | RING | |
| WT CONTAINER + WET SOIL (gm) | 329.96 | 304.82 | 293.61 | 293.61 |
| WT CONTAINER + DRY SOIL (gm) | 249.87 | 264.14 | 264.14 | 264.14 |
| WT CONTAINER (gm) | 91.58 | 183.75 | 183.75 | 183.75 |
| WT DRY SOIL (gm) | 158.29 | 80.39 | 80.39 | 80.39 |
| WATER CONTENT (%) | 50.60 | 50.60 | 36.66 | 36.66 |
| VOID RATIO | ------ | 1.31 | 0.93 | ------ |
| DEGREE OF SATURATION (%) | ------ | 98.67 | 99.99 | ------ |
| DRY DENSITY (lb/ft^3) | ------ | 68.98 | 82.27 | ------ |

URS

# CONSOLIDATION TEST
# TIME CURVES (STEP 3 OF 9)
# STRESS : 0.5 (t/ft^2)





Woodward Clyde Consultants
Project Name : L. Guadalupe River Flood
Project No : 9500112030 Boring No : EB-27 Sample No : 5
Test Date : 10/24/01 Test No : EB27-5 Depth : 18-20 feet
Description : Gray silty Clay with blk carbon & organic peat

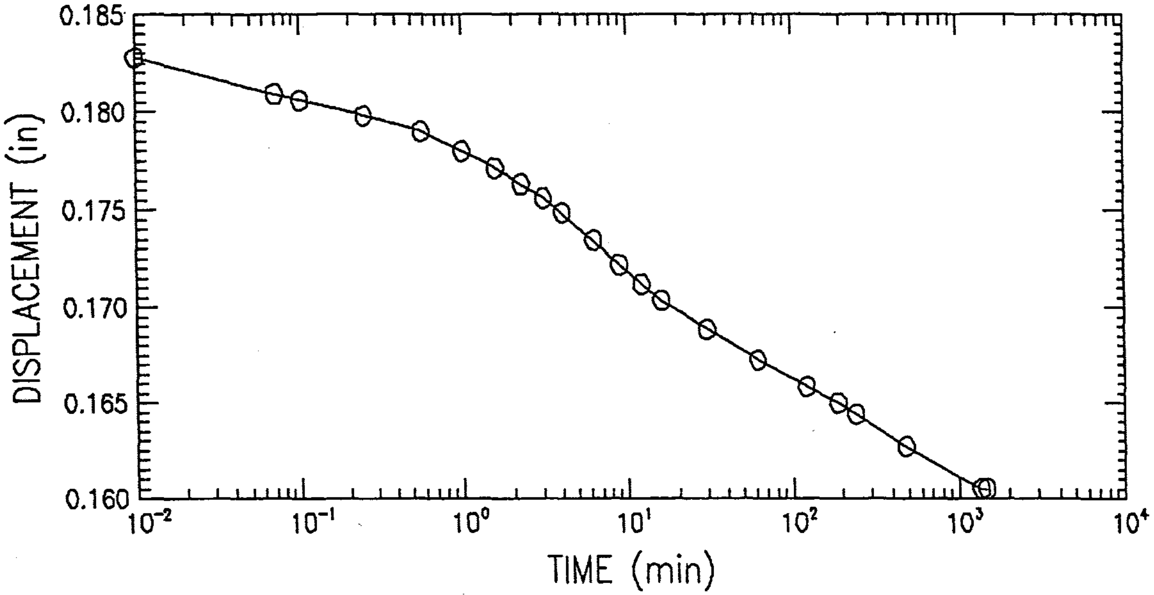
# CONSOLIDATION TEST DATA

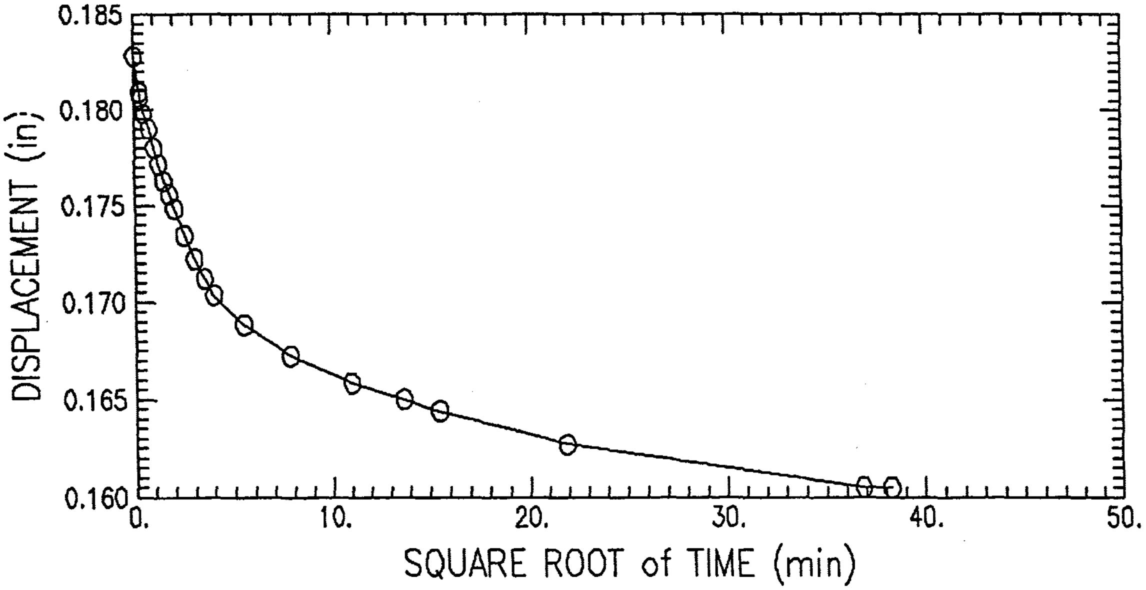
Project : L. Guadalupe River Flood  Location : San Jose, CA  Project No. : 9500112030
Test No. : EB27-5
Boring No. : EB-27  Test Date : 10/24/01  Tested by : C. Wason
Sample No. : 5  Sample Type : Shelby  Depth : 18-20 feet
Checked by : S. Capps
Soil Description : Gray silty Clay with blk carbon & organic peat
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

Load Increment : 3/9
Stress increment from 0.25 (t/ft^2) to 0.50 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.299 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0012 | 1.296 | 0.13 |
| 3) | | | 0.10 | 0.32 | 0.0014 | 1.296 | 0.15 |
| 4) | | | 0.25 | 0.50 | 0.0020 | 1.294 | 0.20 |
| 5) | | | 0.57 | 0.75 | 0.0025 | 1.293 | 0.26 |
| 6) | | | 1.00 | 1.00 | 0.0030 | 1.292 | 0.31 |
| 7) | | | 1.57 | 1.25 | 0.0034 | 1.291 | 0.36 |
| 8) | | | 2.25 | 1.50 | 0.0038 | 1.290 | 0.40 |
| 9) | | | 3.07 | 1.75 | 0.0042 | 1.289 | 0.44 |
| 10) | | | 4.00 | 2.00 | 0.0046 | 1.288 | 0.49 |
| 11) | | | 6.25 | 2.50 | 0.0053 | 1.286 | 0.55 |
| 12) | | | 9.00 | 3.00 | 0.0060 | 1.285 | 0.63 |
| 13) | | | 12.25 | 3.50 | 0.0065 | 1.283 | 0.68 |
| 14) | | | 16.00 | 4.00 | 0.0068 | 1.283 | 0.72 |
| 15) | | | 30.00 | 5.48 | 0.0075 | 1.281 | 0.78 |
| 16) | | | 73.00 | 8.54 | 0.0083 | 1.279 | 0.87 |
| 17) | | | 120.00 | 10.95 | 0.0086 | 1.278 | 0.89 |
| 18) | | | 160.00 | 12.65 | 0.0089 | 1.278 | 0.93 |
| 19) | | | 1025.00 | 32.02 | 0.0100 | 1.275 | 1.04 |
| 20) | | | 1093.00 | 33.06 | 0.0100 | 1.275 | 1.04 |

# CONSOLIDATION TEST
# TIME CURVES (STEP 4 OF 9)
# STRESS : 1 (t/ft^2)





Woodward Clyde Consultants
Project Name : L. Guadalupe River Flood
Project No : 9500112030 Boring No : EB-27 Sample No : 5
Test Date : 10/24/01 Test No : EB27-5 Depth : 18-20 feet
Description : Gray silty Clay with blk carbon & organic peat

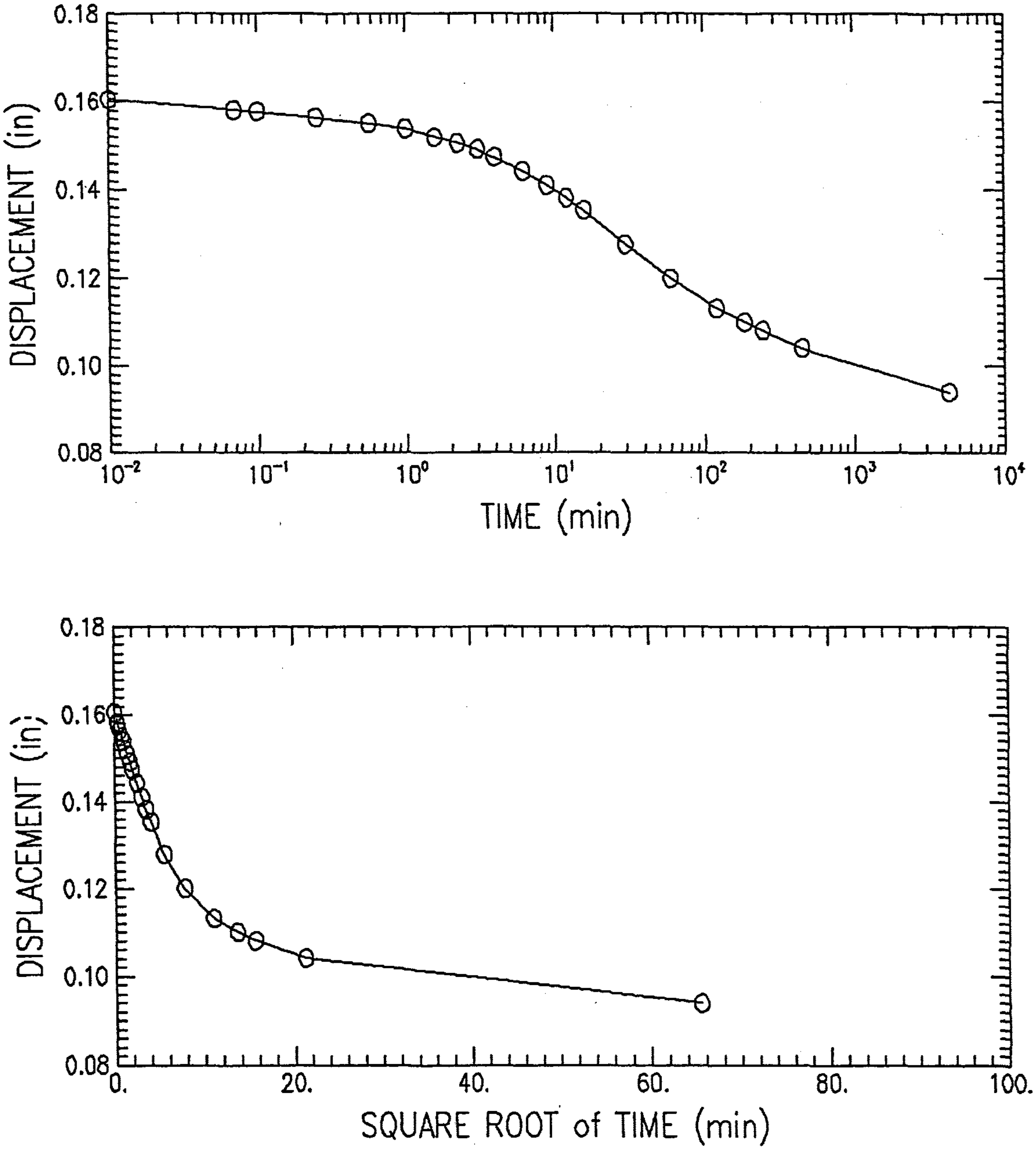
## CONSOLIDATION TEST DATA

Project : L. Guadalupe River Flood Location : San Jose, CA Project No. : 9500112030
Test No. : EB27-5
Boring No. : EB-27 Test Date : 10/24/01 Tested by : C. Wason
Sample No. : 5 Sample Type : Shelby Depth : 18-20 feet
Checked by : S. Capps
Soil Description : Gray silty Clay with blk carbon & organic peat
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

Load Increment : 4/9
Stress increment from 0.50 (t/ft^2) to 1.00 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.283 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0019 | 1.279 | 0.20 |
| 3) | | | 0.10 | 0.32 | 0.0022 | 1.278 | 0.23 |
| 4) | | | 0.25 | 0.50 | 0.0030 | 1.276 | 0.31 |
| 5) | | | 0.57 | 0.75 | 0.0038 | 1.274 | 0.40 |
| 6) | | | 1.00 | 1.00 | 0.0048 | 1.272 | 0.50 |
| 7) | | | 1.57 | 1.25 | 0.0056 | 1.270 | 0.59 |
| 8) | | | 2.25 | 1.50 | 0.0065 | 1.268 | 0.68 |
| 9) | | | 3.07 | 1.75 | 0.0072 | 1.266 | 0.75 |
| 10) | | | 4.00 | 2.00 | 0.0079 | 1.264 | 0.83 |
| 11) | | | 6.25 | 2.50 | 0.0093 | 1.261 | 0.98 |
| 12) | | | 9.00 | 3.00 | 0.0106 | 1.258 | 1.11 |
| 13) | | | 12.25 | 3.50 | 0.0116 | 1.255 | 1.21 |
| 14) | | | 16.00 | 4.00 | 0.0124 | 1.253 | 1.30 |
| 15) | | | 30.00 | 5.48 | 0.0139 | 1.250 | 1.46 |
| 16) | | | 61.00 | 7.81 | 0.0155 | 1.246 | 1.62 |
| 17) | | | 121.00 | 11.00 | 0.0169 | 1.242 | 1.77 |
| 18) | | | 188.00 | 13.71 | 0.0178 | 1.240 | 1.86 |
| 19) | | | 240.00 | 15.49 | 0.0184 | 1.239 | 1.92 |
| 20) | | | 480.00 | 21.91 | 0.0201 | 1.235 | 2.10 |
| 21) | | | 1360.00 | 36.88 | 0.0223 | 1.229 | 2.33 |
| 22) | | | 1465.00 | 38.28 | 0.0223 | 1.229 | 2.33 |

# CONSOLIDATION TEST
## TIME CURVES (STEP 5 OF 9)
## STRESS : 2 (t/ft^2)



Woodward Clyde Consultants
Project Name : L. Guadalupe River Flood
Project No : 9500112030 Boring No : EB-27 Sample No : 5
Test Date : 10/24/01 Test No : EB27-5 Depth : 18-20 feet
Description : Gray silty Clay with blk carbon & organic peat

## CONSOLIDATION TEST DATA

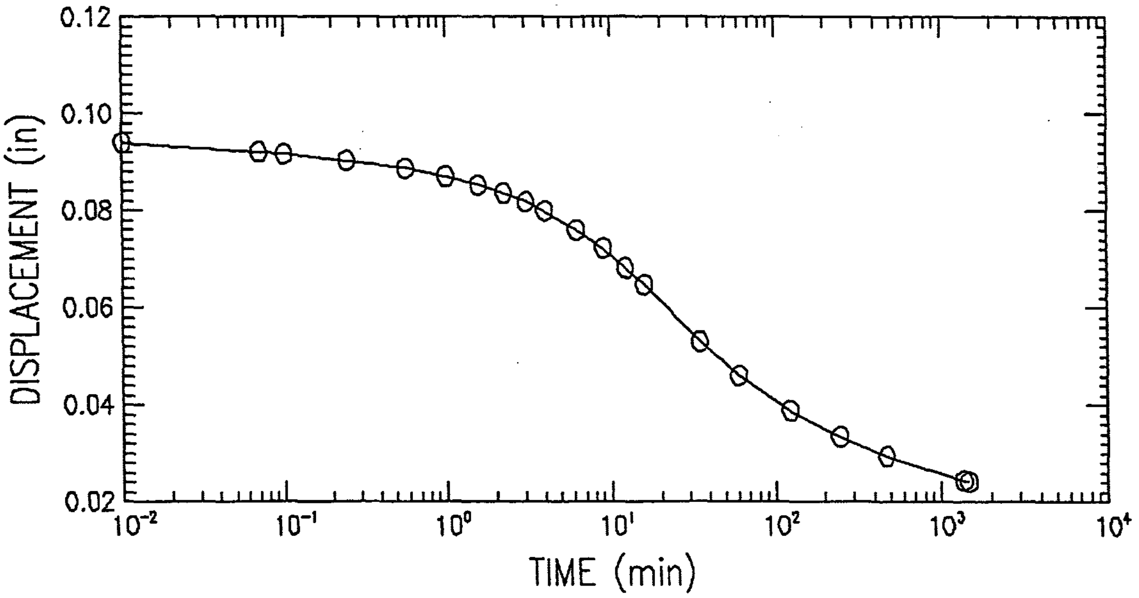
Project : L. Guadalupe River Flood  Location : San Jose, CA  Project No. : 9500112030
Test No. : EB27-5
Boring No. : EB-27  Test Date : 10/24/01  Tested by : C. Wason
Sample No. : 5  Sample Type : Shelby  Depth : 18-20 feet
Checked by : S. Capps
Soil Description : Gray silty Clay with blk carbon & organic peat
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

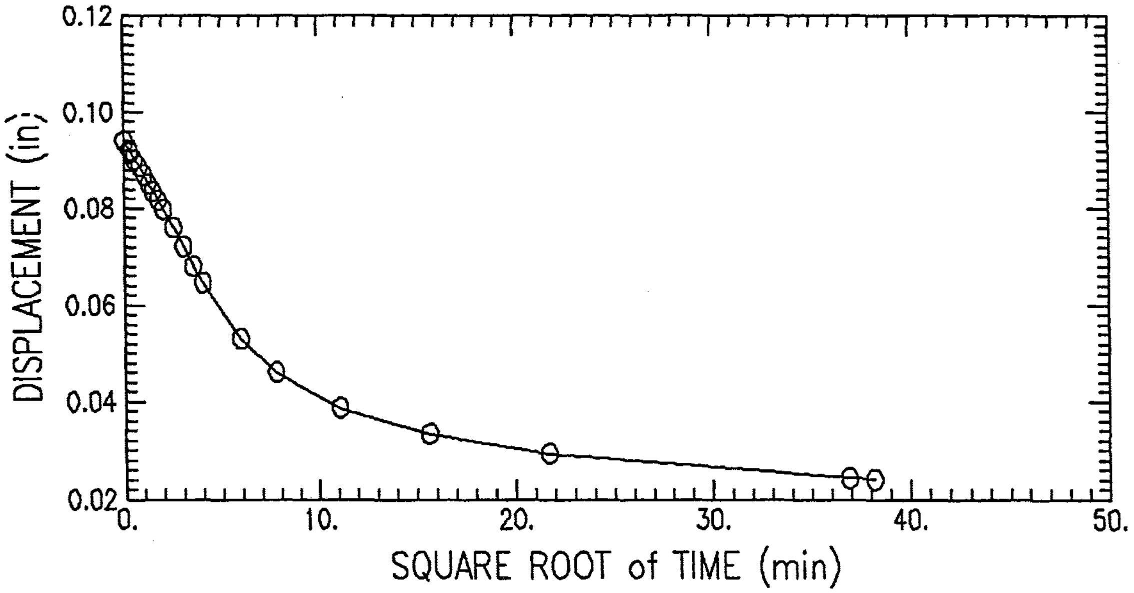
Load Increment : 5/9
Stress increment from 1.00 (t/ft^2) to 2.00 (t/ft^2)

|  | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) |  |  | 0.00 | 0.00 | 0.0000 | 1.254 | 0.00 |
| 2) |  |  | 0.07 | 0.26 | 0.0023 | 1.248 | 0.24 |
| 3) |  |  | 0.10 | 0.32 | 0.0027 | 1.247 | 0.28 |
| 4) |  |  | 0.25 | 0.50 | 0.0041 | 1.244 | 0.43 |
| 5) |  |  | 0.57 | 0.75 | 0.0054 | 1.241 | 0.56 |
| 6) |  |  | 1.00 | 1.00 | 0.0066 | 1.238 | 0.69 |
| 7) |  |  | 1.57 | 1.25 | 0.0085 | 1.233 | 0.89 |
| 8) |  |  | 2.25 | 1.50 | 0.0098 | 1.230 | 1.02 |
| 9) |  |  | 3.07 | 1.75 | 0.0113 | 1.226 | 1.18 |
| 10) |  |  | 4.00 | 2.00 | 0.0131 | 1.222 | 1.37 |
| 11) |  |  | 6.25 | 2.50 | 0.0162 | 1.215 | 1.69 |
| 12) |  |  | 9.00 | 3.00 | 0.0194 | 1.207 | 2.03 |
| 13) |  |  | 12.25 | 3.50 | 0.0223 | 1.200 | 2.33 |
| 14) |  |  | 16.00 | 4.00 | 0.0251 | 1.193 | 2.62 |
| 15) |  |  | 30.00 | 5.48 | 0.0327 | 1.175 | 3.42 |
| 16) |  |  | 60.00 | 7.75 | 0.0405 | 1.156 | 4.23 |
| 17) |  |  | 120.00 | 10.95 | 0.0473 | 1.140 | 4.94 |
| 18) |  |  | 184.00 | 13.56 | 0.0504 | 1.132 | 5.26 |
| 19) |  |  | 243.00 | 15.59 | 0.0524 | 1.127 | 5.47 |
| 20) |  |  | 450.00 | 21.21 | 0.0564 | 1.118 | 5.89 |
| 21) |  |  | 4287.00 | 65.48 | 0.0665 | 1.093 | 6.95 |

**URS**

# CONSOLIDATION TEST
## TIME CURVES (STEP 6 OF 9)
## STRESS : 4 (t/ft^2)





Woodward Clyde Consultants
Project Name : L. Guadalupe River Flood
Project No : 9500112030 Boring No : EB-27 Sample No : 5
Test Date : 10/24/01 Test No : EB27-5 Depth : 18-20 feet
Description : Gray silty Clay with blk carbon & organic peat

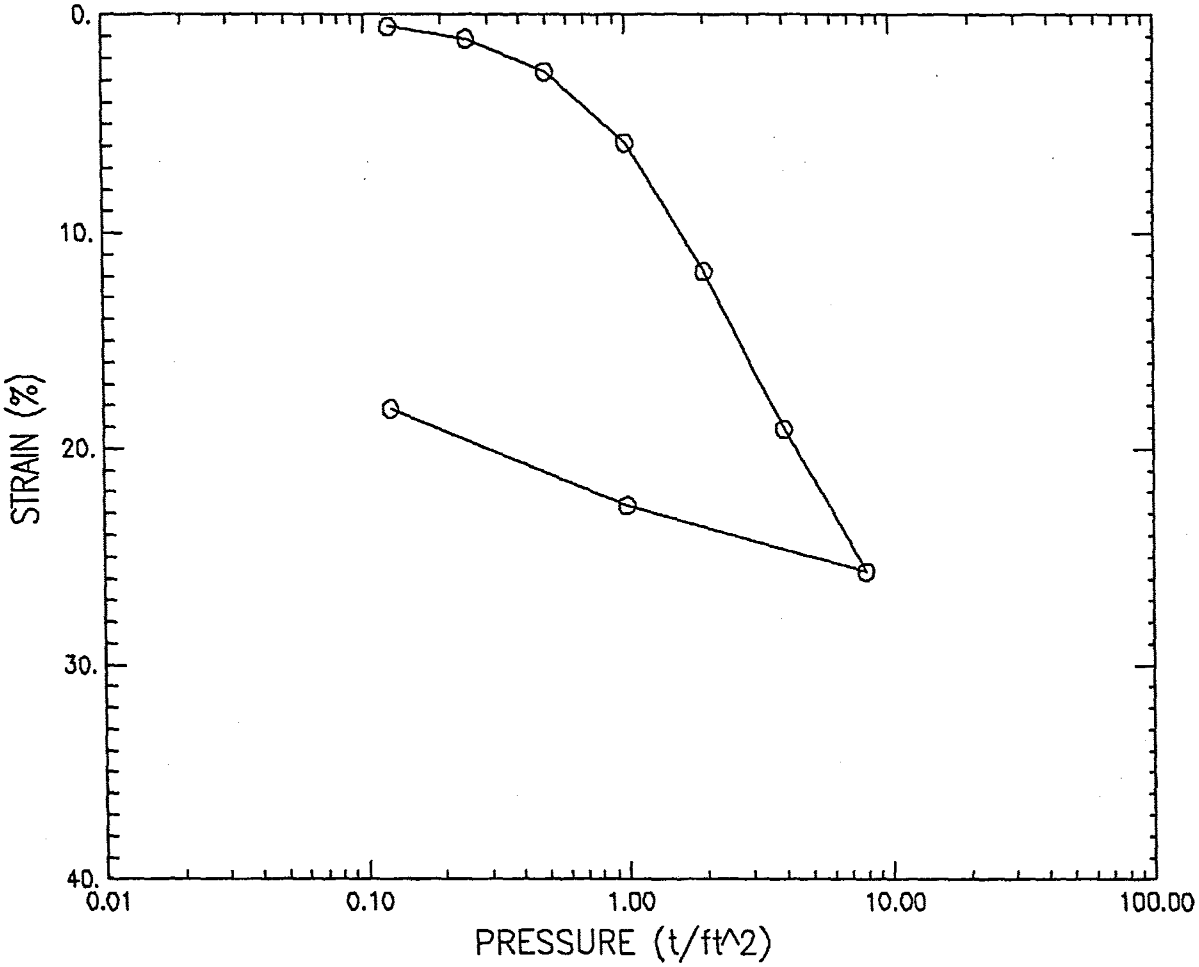
Fri Nov 02 16:51:51 2001

# CONSOLIDATION TEST DATA

Project : L. Guadalupe River Flood Location : San Jose, CA Project No. : 9500112030
Test No. : EB27-5
Boring No. : EB-27 Test Date : 10/24/01 Tested by : C. Wason
Sample No. : 5 Sample Type : Shelby Depth : 18-20 feet
Checked by : S. Capps
Soil Description : Gray silty Clay with blk carbon & organic peat
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

Load Increment : 6/9
Stress increment from 2.00 (t/ft^2) to 4.00 (t/ft^2)

|  | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) |  |  | 0.00 | 0.00 | 0.0000 | 1.147 | 0.00 |
| 2) |  |  | 0.07 | 0.26 | 0.0019 | 1.142 | 0.20 |
| 3) |  |  | 0.10 | 0.32 | 0.0023 | 1.141 | 0.24 |
| 4) |  |  | 0.25 | 0.50 | 0.0037 | 1.138 | 0.38 |
| 5) |  |  | 0.57 | 0.75 | 0.0052 | 1.134 | 0.54 |
| 6) |  |  | 1.00 | 1.00 | 0.0069 | 1.130 | 0.72 |
| 7) |  |  | 1.57 | 1.25 | 0.0087 | 1.126 | 0.91 |
| 8) |  |  | 2.25 | 1.50 | 0.0104 | 1.122 | 1.09 |
| 9) |  |  | 3.07 | 1.75 | 0.0121 | 1.118 | 1.27 |
| 10) |  |  | 4.00 | 2.00 | 0.0141 | 1.113 | 1.47 |
| 11) |  |  | 6.25 | 2.50 | 0.0180 | 1.104 | 1.88 |
| 12) |  |  | 9.00 | 3.00 | 0.0218 | 1.094 | 2.28 |
| 13) |  |  | 12.25 | 3.50 | 0.0259 | 1.085 | 2.71 |
| 14) |  |  | 16.00 | 4.00 | 0.0293 | 1.077 | 3.06 |
| 15) |  |  | 35.00 | 5.92 | 0.0409 | 1.048 | 4.27 |
| 16) |  |  | 60.00 | 7.75 | 0.0479 | 1.032 | 5.00 |
| 17) |  |  | 123.00 | 11.09 | 0.0553 | 1.014 | 5.77 |
| 18) |  |  | 245.00 | 15.65 | 0.0606 | 1.001 | 6.33 |
| 19) |  |  | 472.00 | 21.73 | 0.0646 | 0.991 | 6.75 |
| 20) |  |  | 1363.00 | 36.92 | 0.0696 | 0.979 | 7.27 |
| 21) |  |  | 1460.00 | 38.21 | 0.0698 | 0.979 | 7.29 |



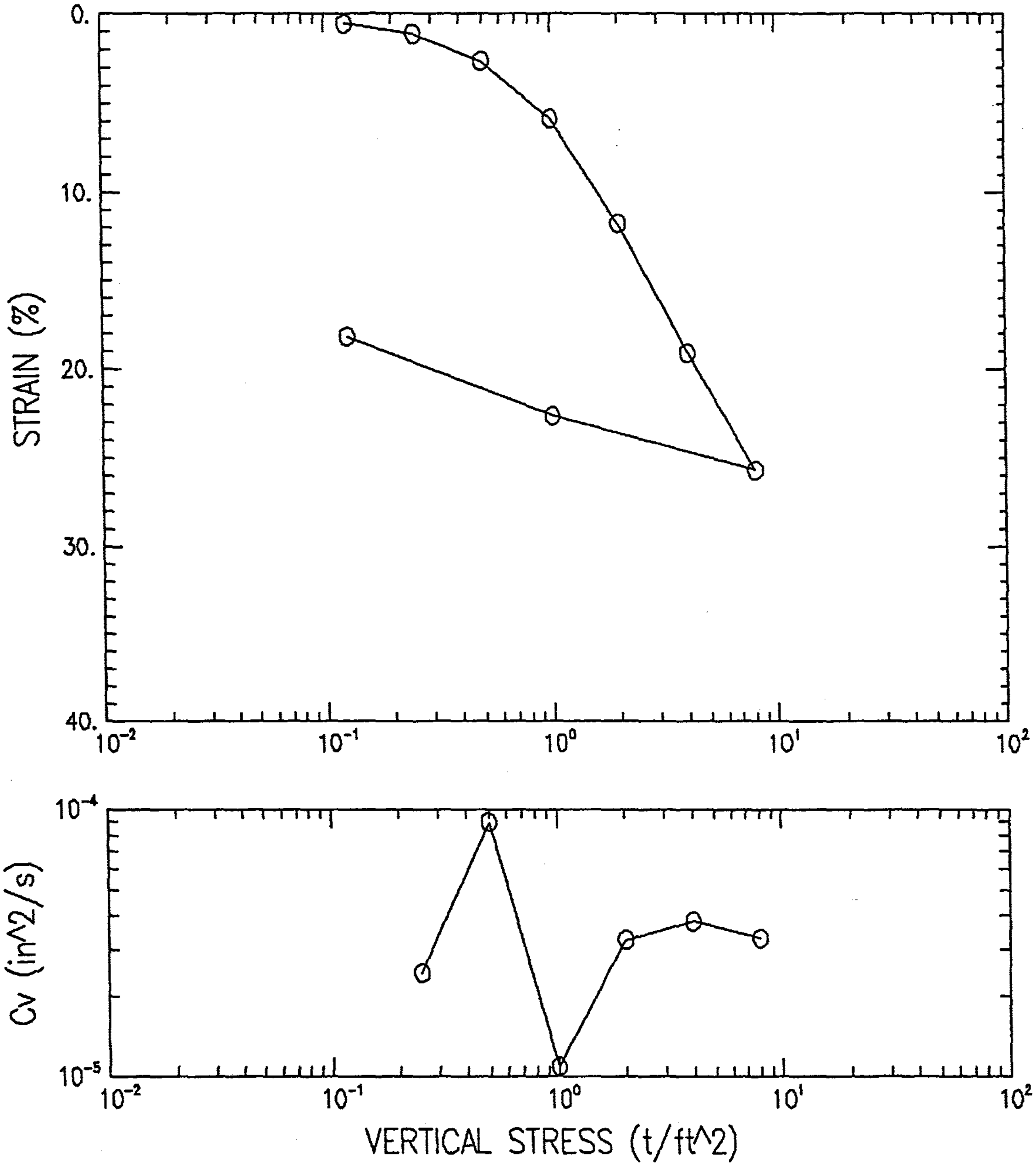
| | | | BEFORE TEST | AFTER TEST |
|---|---|---|---|---|
| OVERBURDEN PRESSURE (t/ft^2) | | WATER CONTENT (%) | 53.651 | 38.330 |
| PRECONSOL. PRESSURE (t/ft^2) | | DRY DENSITY (lb/ft^3) | 67.807 | 82.834 |
| COMPRESSION INDEX | | SATURATION (%) | 97.495 | 100.003 |
| TYPE SPECIMEN | | VOID RATIO | 1.486 | 1.035 |
| DIA. (in) 2.429 | HT. (in) 0.948 | BACK PRESSURE (t/ft^2) | | |

| | | | | |
|---|---|---|---|---|
| CLASSIFICATION Dark Gray silty Clay | | | | |
| LL 0.0 | PL 0.0 | PI 0.0 | PROJECT L. Guadalupe River Flood | |
| GS 2.700 | $D_{10}$ | | Data File: EB27-9.CON | |
| REMARKS | | | BORING NO. EB-27 | SAMPLE NO. 9 |
| | | | DEPTH 41.5-44 ft | DATE 10/24/01 |

Woodward Clyde Consultants
CONSOLIDATION TEST REPORT

URS

# CONSOLIDATION TEST
# SUMMARY REPORT



Woodward Clyde Consultants
Project Name : L. Guadalupe River Flood
Project No : 9500112030 Boring No : EB-27 Sample No : 9
Test Date : 10/24/01 Test No : EB27-9 Depth : 41.5-44 ft
Description : Dark Gray silty Clay

Fri Nov 02 16:55:48 2001

## CONSOLIDATION TEST DATA

Project : L. Guadalupe River Flood    Location : San Jose, CA    Project No. : 9500112030
Test No. : EB27-9
Boring No. : EB-27    Test Date : 10/24/01    Tested by : C. Wason
Sample No. : 9    Sample Type : Shelby    Depth : 41.5-44 ft
Checked by : S. Capps
Soil Description : Dark Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.429 inches in diameter

| | APPLIED PRESSURE (t/ft^2) | FINAL DISPLACEMENT (in) | VOID RATIO | STRAIN AT END (%) | FITTING TIME (min) | | COEFFICIENT OF CONSOLIDATION (in^2/s) | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | SQ.RT. | LOG | SQ.RT. | LOG | AVE |
| 1) | 0.13 | 0.005 | 1.472 | 0.54 | 0.0 | 0.0 | 0.00E+000 | 0.00E+000 | 0.00E+000 |
| 2) | 0.25 | 0.011 | 1.458 | 1.11 | 29.7 | 0.0 | 2.44E-005 | 0.00E+000 | 0.00E+000 |
| 3) | 0.50 | 0.025 | 1.421 | 2.62 | 7.9 | 0.0 | 8.95E-005 | 0.00E+000 | 0.00E+000 |
| 4) | 1.00 | 0.055 | 1.341 | 5.84 | 62.2 | 0.0 | 1.09E-005 | 0.00E+000 | 0.00E+000 |
| 5) | 2.00 | 0.111 | 1.193 | 11.76 | 18.9 | 0.0 | 3.24E-005 | 0.00E+000 | 0.00E+000 |
| 6) | 4.00 | 0.181 | 1.012 | 19.08 | 13.9 | 0.0 | 3.81E-005 | 0.00E+000 | 0.00E+000 |
| 7) | 8.00 | 0.244 | 0.847 | 25.69 | 13.5 | 0.0 | 3.29E-005 | 0.00E+000 | 0.00E+000 |
| 8) | 1.00 | 0.214 | 0.924 | 22.61 | 0.0 | 0.0 | 0.00E+000 | 0.00E+000 | 0.00E+000 |
| 9) | 0.13 | 0.172 | 1.034 | 18.16 | 0.0 | 0.0 | 0.00E+000 | 0.00E+000 | 0.00E+000 |

## CONSOLIDATION TEST DATA

Project : L. Guadalupe River Flood Location : San Jose, CA Project No. : 9500112030
Test No. : EB27-9
Boring No. : EB-27 Test Date : 10/24/01 Tested by : C. Wason
Sample No. : 9 Sample Type : Shelby Depth : 41.5-44 ft
Checked by : S. Capps
Soil Description : Dark Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.429 inches in diameter

Liquid Limit : 0 Plastic Limit : 0 Plasticity Index : 0
Specific Gravity : 2.70 Initial Height : 0.95 Sample Diameter : 2.43 (in)
Initial Void Ratio : 1.481 Weight of Piston and top cap : 0.00 (gm)

| | TRIMMINGS | SPECIMEN + RING BEFORE | SPECIMEN + RING AFTER | AFTER |
|---|---|---|---|---|
| CONTAINER NO. | | RING | RING | |
| WT CONTAINER + WET SOIL (gm) | 271.80 | 309.72 | 297.74 | 297.74 |
| WT CONTAINER + DRY SOIL (gm) | 208.16 | 267.77 | 267.77 | 267.77 |
| WT CONTAINER (gm) | 89.53 | 189.58 | 189.58 | 189.58 |
| WT DRY SOIL (gm) | 118.63 | 78.19 | 78.19 | 78.19 |
| WATER CONTENT (%) | 53.65 | 53.65 | 38.33 | 38.33 |
| VOID RATIO | ------ | 1.49 | 1.03 | ------ |
| DEGREE OF SATURATION (%) | ------ | 97.50 | 100.00 | ------ |
| DRY DENSITY (lb/ft^3) | ------ | 67.81 | 82.83 | ------ |

Fri Nov 02 17:20:37 2001

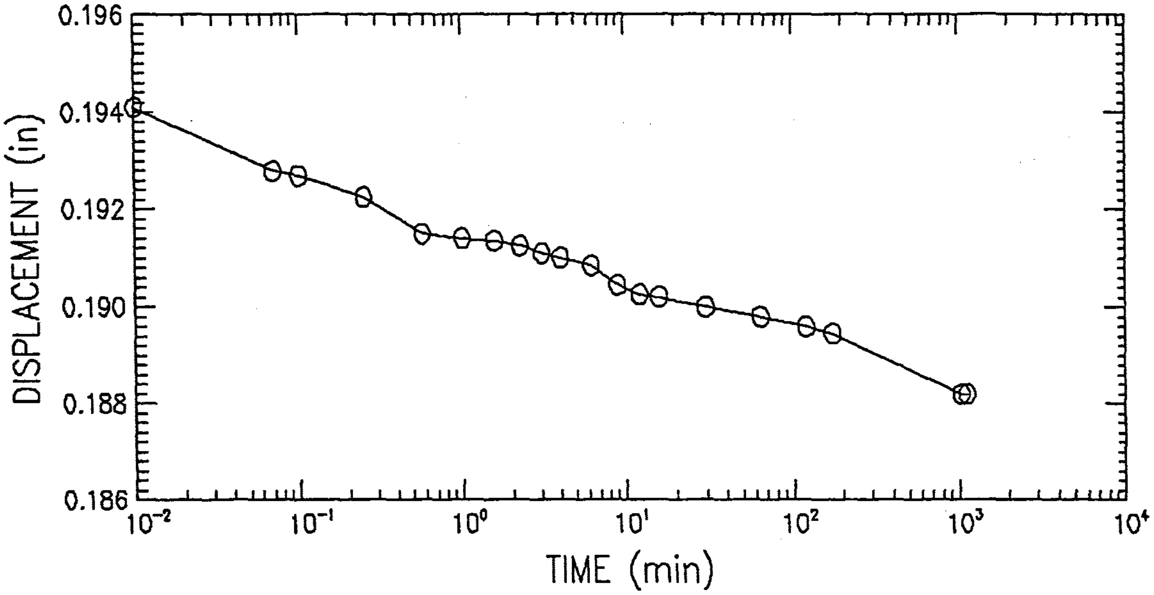
## CONSOLIDATION TEST DATA

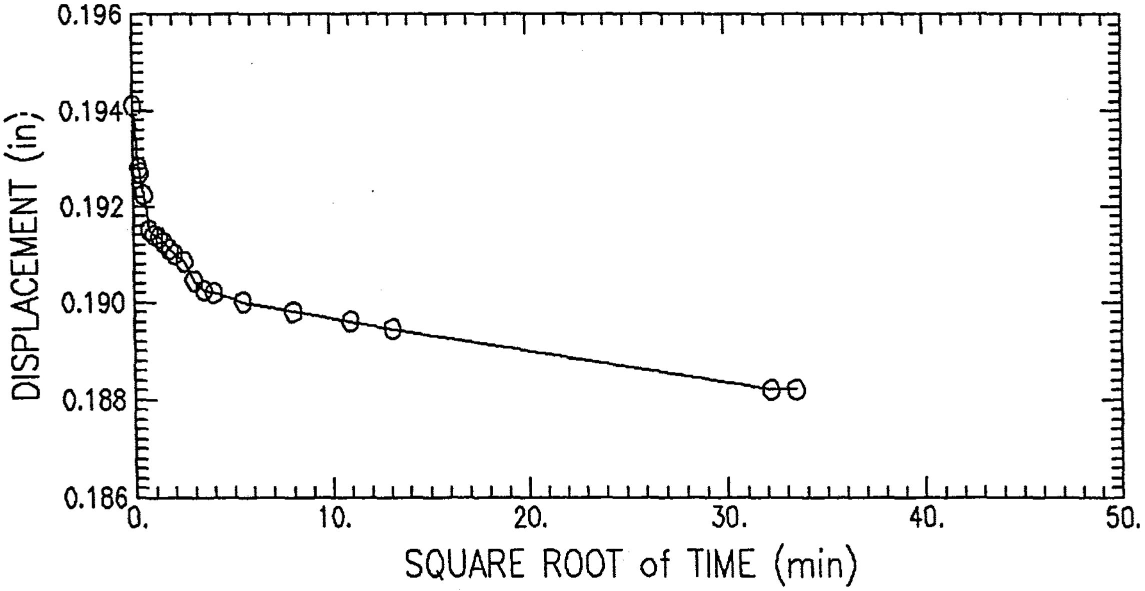
Project : L. Guadalupe River Flood
Location : San Jose, CA
Project No. : 9500112030
Test No. : EB27-9
Boring No. : EB-27
Test Date : 10/24/01
Tested by : C. Wason
Sample No. : 9
Sample Type : Shelby
Depth : 41.5-44 ft
Checked by : S. Capps
Soil Description : Dark Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.429 inches in diameter

Load Increment : 2/9
Stress increment from 0.13 (t/ft^2) to 0.25 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.466 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0013 | 1.462 | 0.14 |
| 3) | | | 0.10 | 0.32 | 0.0014 | 1.462 | 0.15 |
| 4) | | | 0.25 | 0.50 | 0.0018 | 1.461 | 0.20 |
| 5) | | | 0.57 | 0.75 | 0.0026 | 1.459 | 0.27 |
| 6) | | | 1.00 | 1.00 | 0.0027 | 1.458 | 0.28 |
| 7) | | | 1.57 | 1.25 | 0.0028 | 1.458 | 0.29 |
| 8) | | | 2.25 | 1.50 | 0.0028 | 1.458 | 0.30 |
| 9) | | | 3.07 | 1.75 | 0.0030 | 1.458 | 0.32 |
| 10) | | | 4.00 | 2.00 | 0.0031 | 1.457 | 0.33 |
| 11) | | | 6.25 | 2.50 | 0.0033 | 1.457 | 0.34 |
| 12) | | | 9.00 | 3.00 | 0.0036 | 1.456 | 0.39 |
| 13) | | | 12.25 | 3.50 | 0.0038 | 1.455 | 0.41 |
| 14) | | | 16.00 | 4.00 | 0.0039 | 1.455 | 0.41 |
| 15) | | | 30.00 | 5.48 | 0.0041 | 1.455 | 0.43 |
| 16) | | | 64.00 | 8.00 | 0.0043 | 1.454 | 0.45 |
| 17) | | | 120.00 | 10.95 | 0.0045 | 1.454 | 0.47 |
| 18) | | | 172.00 | 13.11 | 0.0046 | 1.453 | 0.49 |
| 19) | | | 1037.00 | 32.20 | 0.0059 | 1.450 | 0.62 |
| 20) | | | 1120.00 | 33.47 | 0.0059 | 1.450 | 0.62 |

# CONSOLIDATION TEST
# TIME CURVES (STEP 2 OF 9)
# STRESS : 0.25 (t/ft^2)

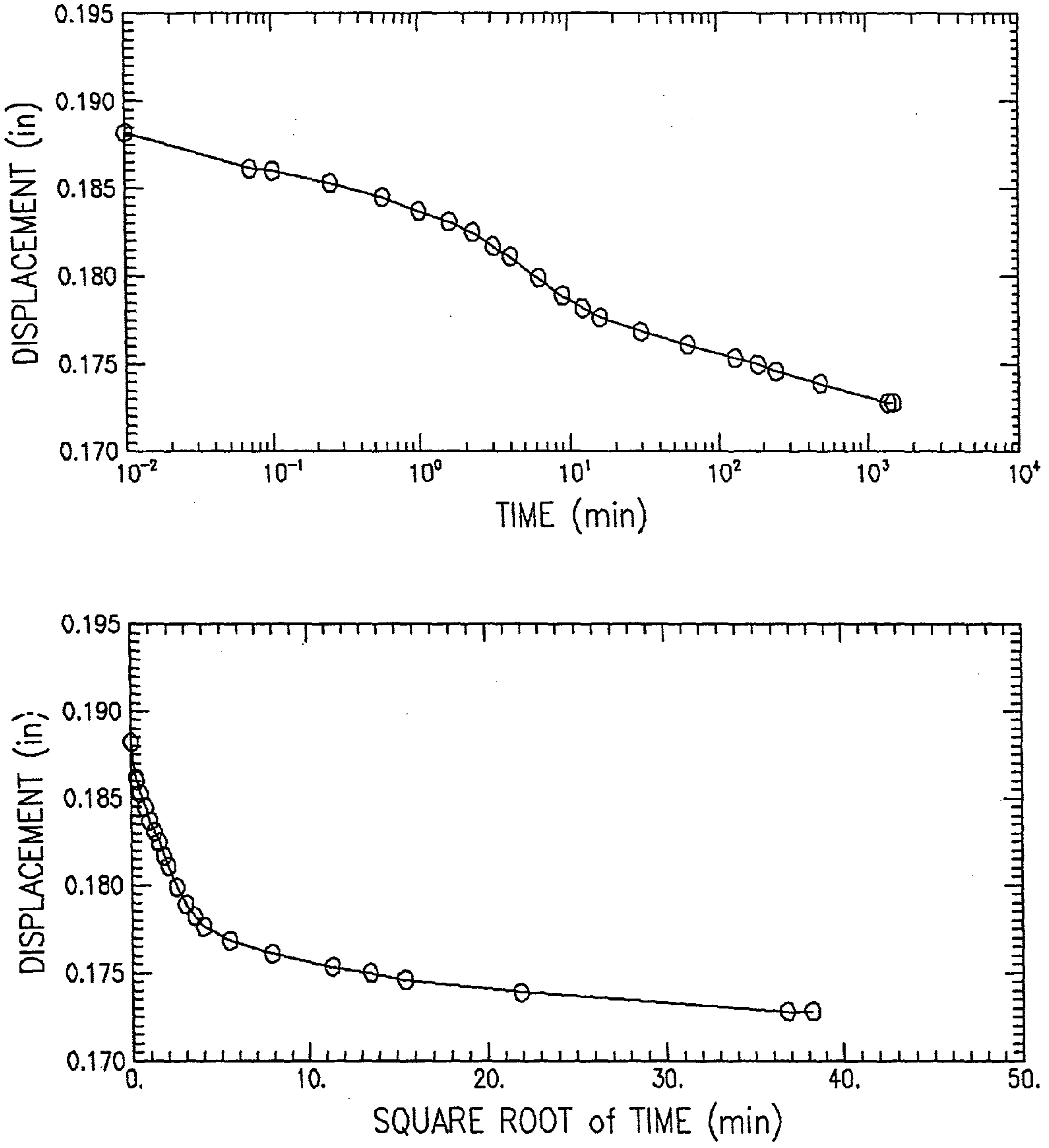




Woodward Clyde Consultants
Project Name : L. Guadalupe River Flood
Project No : 9500112030 Boring No : EB-27 Sample No : 9
Test Date : 10/24/01 Test No : EB27-9 Depth : 41.5-44 ft
Description : Dark Gray silty Clay

**URS**

# CONSOLIDATION TEST
## TIME CURVES (STEP 3 OF 9)
## STRESS : 0.5 (t/ft^2)



Woodward Clyde Consultants
Project Name : L. Guadalupe River Flood
Project No : 9500112030 Boring No : EB-27 Sample No : 9
Test Date : 10/24/01 Test No : EB27-9 Depth : 41.5-44 ft
Description : Dark Gray silty Clay

Fri Nov 02 17:21:05 2001

## CONSOLIDATION TEST DATA

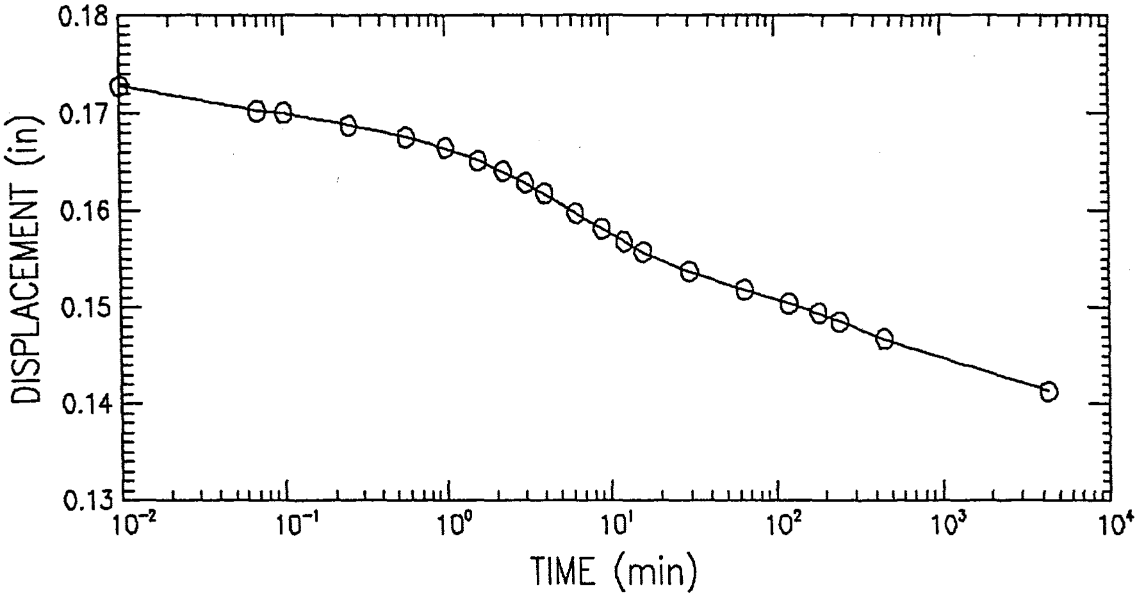
Project : L. Guadalupe River Flood
Location : San Jose, CA
Project No. : 9500112030
Test No. : EB27-9
Boring No. : EB-27
Test Date : 10/24/01
Tested by : C. Wason
Sample No. : 9
Sample Type : Shelby
Depth : 41.5-44 ft
Checked by : S. Capps
Soil Description : Dark Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.429 inches in diameter

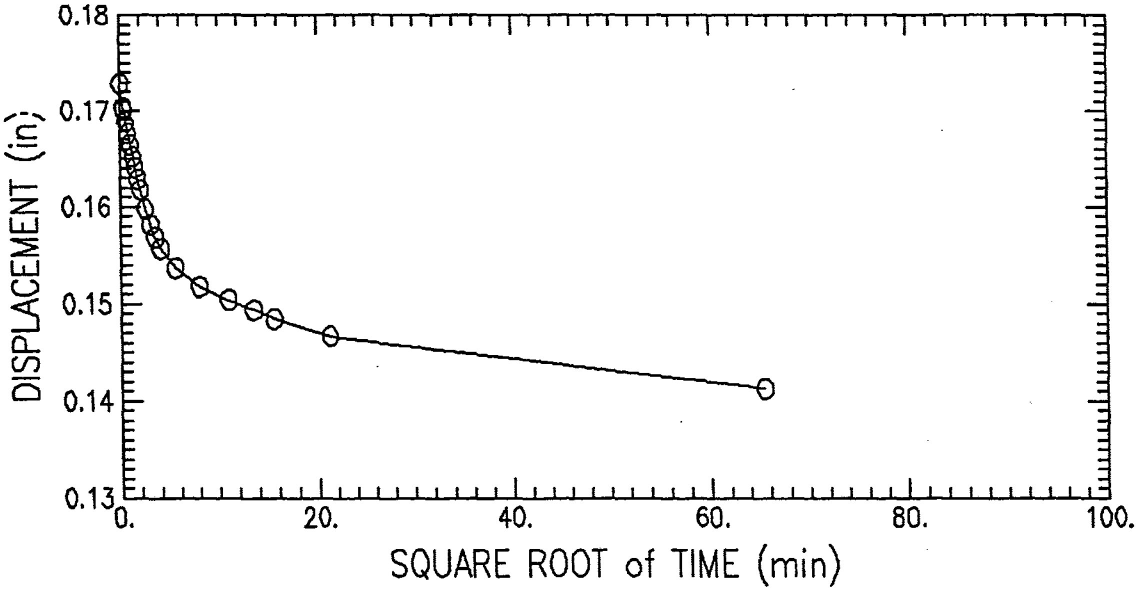
Load Increment : 3/9
Stress increment from 0.25 (t/ft^2) to 0.50 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.466 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0020 | 1.460 | 0.22 |
| 3) | | | 0.10 | 0.32 | 0.0022 | 1.460 | 0.23 |
| 4) | | | 0.25 | 0.50 | 0.0029 | 1.458 | 0.31 |
| 5) | | | 0.57 | 0.75 | 0.0037 | 1.456 | 0.39 |
| 6) | | | 1.00 | 1.00 | 0.0045 | 1.454 | 0.47 |
| 7) | | | 1.57 | 1.25 | 0.0051 | 1.452 | 0.54 |
| 8) | | | 2.25 | 1.50 | 0.0057 | 1.451 | 0.60 |
| 9) | | | 3.07 | 1.75 | 0.0065 | 1.449 | 0.69 |
| 10) | | | 4.00 | 2.00 | 0.0071 | 1.447 | 0.75 |
| 11) | | | 6.25 | 2.50 | 0.0083 | 1.444 | 0.88 |
| 12) | | | 9.00 | 3.00 | 0.0093 | 1.441 | 0.98 |
| 13) | | | 12.25 | 3.50 | 0.0100 | 1.439 | 1.05 |
| 14) | | | 16.00 | 4.00 | 0.0106 | 1.438 | 1.11 |
| 15) | | | 30.00 | 5.48 | 0.0114 | 1.436 | 1.20 |
| 16) | | | 62.00 | 7.87 | 0.0121 | 1.434 | 1.28 |
| 17) | | | 129.00 | 11.36 | 0.0129 | 1.432 | 1.36 |
| 18) | | | 183.00 | 13.53 | 0.0132 | 1.431 | 1.39 |
| 19) | | | 240.00 | 15.49 | 0.0136 | 1.430 | 1.43 |
| 20) | | | 480.00 | 21.91 | 0.0143 | 1.428 | 1.51 |
| 21) | | | 1355.00 | 36.81 | 0.0154 | 1.425 | 1.62 |
| 22) | | | 1462.00 | 38.24 | 0.0154 | 1.425 | 1.62 |

URS

# CONSOLIDATION TEST
# TIME CURVES (STEP 4 OF 9)
# STRESS : 1 (t/ft^2)





Woodward Clyde Consultants
Project Name : L. Guadalupe River Flood
Project No : 9500112030 Boring No : EB-27 Sample No : 9
Test Date : 10/24/01 Test No : EB27-9 Depth : 41.5-44 ft
Description : Dark Gray silty Clay

## CONSOLIDATION TEST DATA

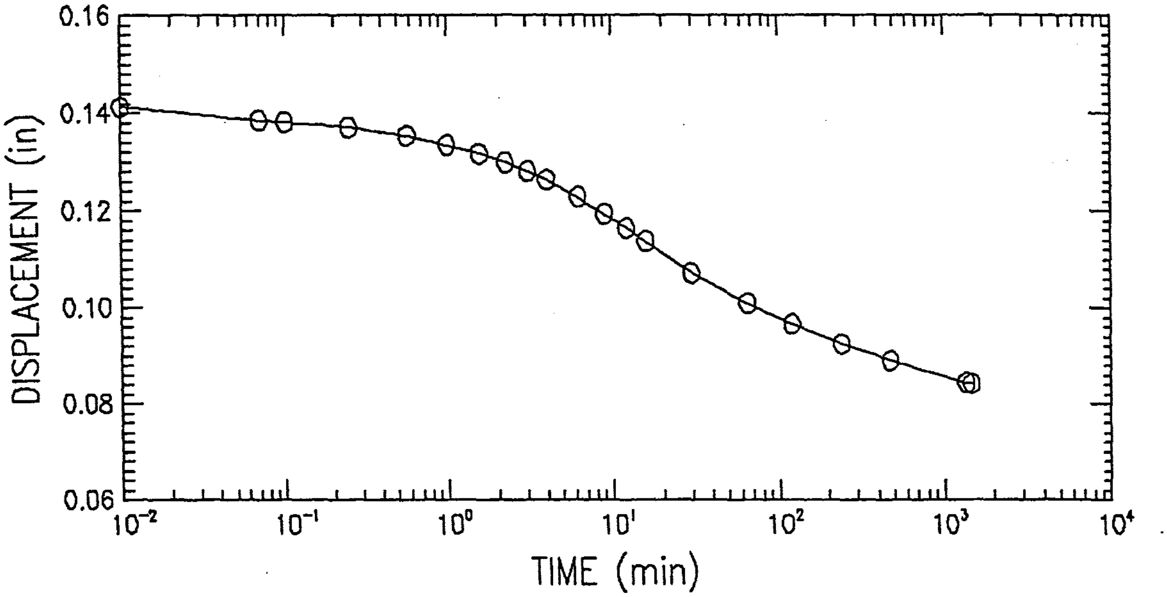
Project : L. Guadalupe River Flood Location : San Jose, CA Project No. : 9500112030
Test No. : EB27-9
Boring No. : EB-27 Test Date : 10/24/01 Tested by : C. Wason
Sample No. : 9 Sample Type : Shelby Depth : 41.5-44 ft
Checked by : S. Capps
Soil Description : Dark Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.429 inches in diameter

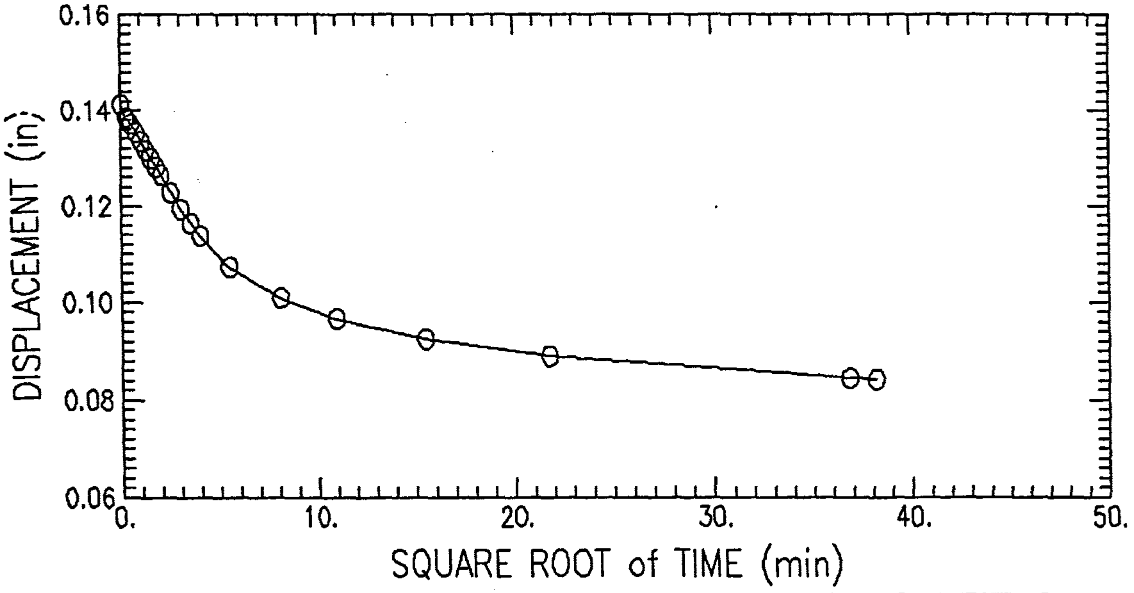
Load Increment : 4/9
Stress increment from 0.50 (t/ft^2) to 1.00 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.441 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0025 | 1.434 | 0.26 |
| 3) | | | 0.10 | 0.32 | 0.0028 | 1.433 | 0.29 |
| 4) | | | 0.25 | 0.50 | 0.0040 | 1.430 | 0.42 |
| 5) | | | 0.57 | 0.75 | 0.0052 | 1.427 | 0.55 |
| 6) | | | 1.00 | 1.00 | 0.0063 | 1.424 | 0.66 |
| 7) | | | 1.57 | 1.25 | 0.0076 | 1.421 | 0.80 |
| 8) | | | 2.25 | 1.50 | 0.0087 | 1.418 | 0.92 |
| 9) | | | 3.07 | 1.75 | 0.0099 | 1.415 | 1.04 |
| 10) | | | 4.00 | 2.00 | 0.0110 | 1.412 | 1.16 |
| 11) | | | 6.25 | 2.50 | 0.0131 | 1.407 | 1.38 |
| 12) | | | 9.00 | 3.00 | 0.0147 | 1.402 | 1.55 |
| 13) | | | 12.25 | 3.50 | 0.0160 | 1.399 | 1.69 |
| 14) | | | 16.00 | 4.00 | 0.0171 | 1.396 | 1.80 |
| 15) | | | 30.00 | 5.48 | 0.0191 | 1.391 | 2.01 |
| 16) | | | 64.00 | 8.00 | 0.0210 | 1.386 | 2.22 |
| 17) | | | 120.00 | 10.95 | 0.0224 | 1.382 | 2.36 |
| 18) | | | 182.00 | 13.49 | 0.0234 | 1.379 | 2.47 |
| 19) | | | 241.00 | 15.52 | 0.0243 | 1.377 | 2.56 |
| 20) | | | 448.00 | 21.17 | 0.0261 | 1.372 | 2.75 |
| 21) | | | 4290.00 | 65.50 | 0.0315 | 1.358 | 3.32 |

**URS**

# CONSOLIDATION TEST
TIME CURVES (STEP 5 OF 9)
STRESS : 2 (t/ft^2)





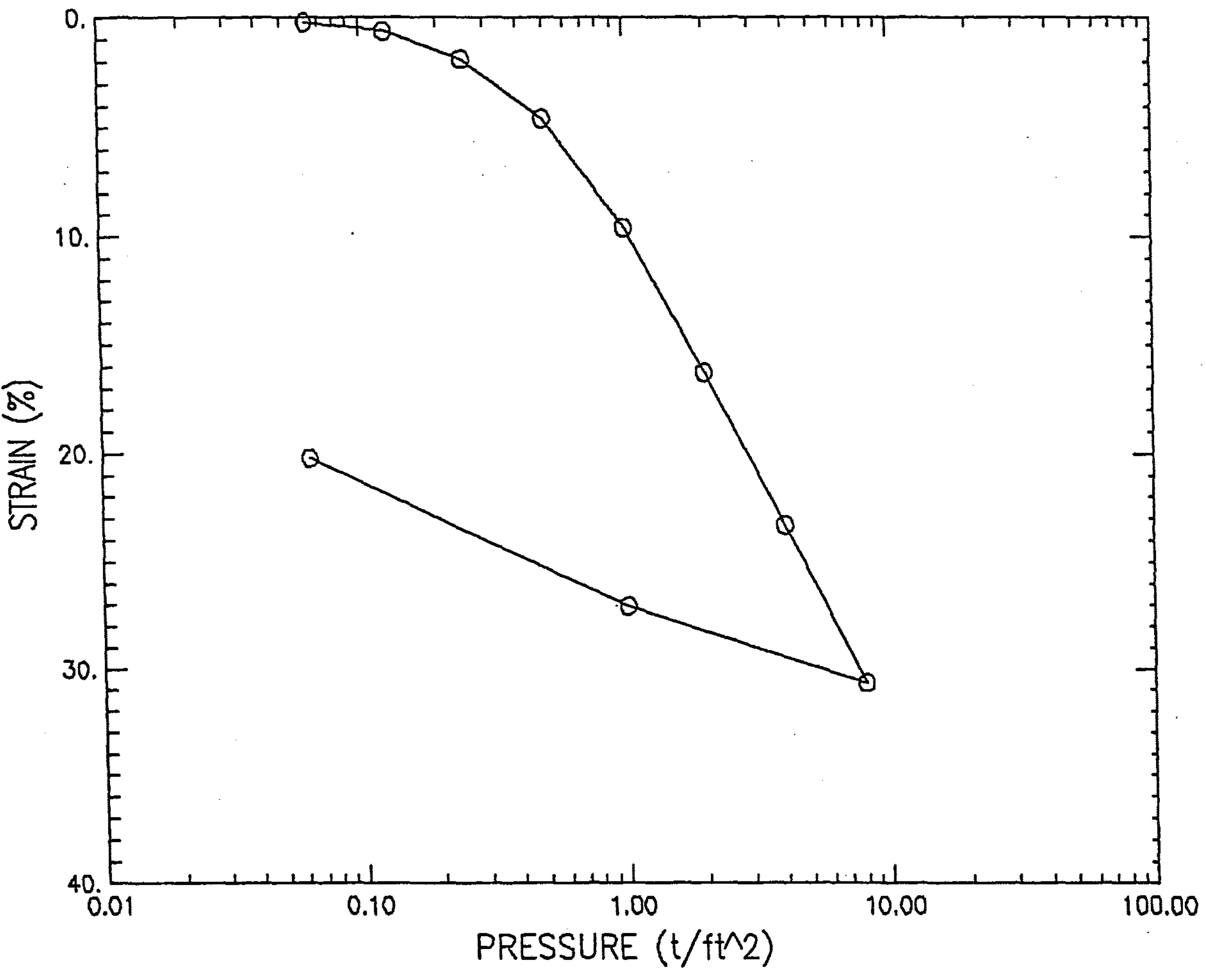
Woodward Clyde Consultants
Project Name : L. Guadalupe River Flood
Project No : 9500112030 Boring No : EB-27 Sample No : 9
Test Date : 10/24/01 Test No : EB27-9 Depth : 41.5-44 ft
Description : Dark Gray silty Clay

# CONSOLIDATION TEST DATA

Project : L. Guadalupe River Flood Location : San Jose, CA Project No. : 9500112030
Test No. : EB27-9
Boring No. : EB-27 Test Date : 10/24/01 Tested by : C. Wason
Sample No. : 9 Sample Type : Shelby Depth : 41.5-44 ft
Checked by : S. Capps
Soil Description : Dark Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.429 inches in diameter

Load Increment : 5/9
Stress increment from 1.00 (t/ft^2) to 2.00 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.399 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0028 | 1.391 | 0.30 |
| 3) | | | 0.10 | 0.32 | 0.0031 | 1.390 | 0.33 |
| 4) | | | 0.25 | 0.50 | 0.0042 | 1.388 | 0.44 |
| 5) | | | 0.57 | 0.75 | 0.0059 | 1.383 | 0.62 |
| 6) | | | 1.00 | 1.00 | 0.0078 | 1.378 | 0.82 |
| 7) | | | 1.57 | 1.25 | 0.0096 | 1.373 | 1.01 |
| 8) | | | 2.25 | 1.50 | 0.0114 | 1.369 | 1.20 |
| 9) | | | 3.07 | 1.75 | 0.0132 | 1.364 | 1.39 |
| 10) | | | 4.00 | 2.00 | 0.0149 | 1.360 | 1.57 |
| 11) | | | 6.25 | 2.50 | 0.0185 | 1.350 | 1.95 |
| 12) | | | 9.00 | 3.00 | 0.0220 | 1.341 | 2.32 |
| 13) | | | 12.25 | 3.50 | 0.0248 | 1.334 | 2.62 |
| 14) | | | 16.00 | 4.00 | 0.0276 | 1.326 | 2.91 |
| 15) | | | 30.00 | 5.48 | 0.0341 | 1.309 | 3.60 |
| 16) | | | 65.00 | 8.06 | 0.0404 | 1.293 | 4.26 |
| 17) | | | 120.00 | 10.95 | 0.0447 | 1.282 | 4.72 |
| 18) | | | 240.00 | 15.49 | 0.0488 | 1.271 | 5.15 |
| 19) | | | 473.00 | 21.75 | 0.0523 | 1.262 | 5.52 |
| 20) | | | 1358.00 | 36.85 | 0.0569 | 1.250 | 6.00 |
| 21) | | | 1460.00 | 38.21 | 0.0571 | 1.249 | 6.02 |



| | | | BEFORE TEST | AFTER TEST |
|---|---|---|---|---|
| OVERBURDEN PRESSURE (t/ft^2) | | WATER CONTENT (%) | 70.016 | 49.102 |
| PRECONSOL. PRESSURE (t/ft^2) | | DRY DENSITY (lb/ft^3) | 58.009 | 72.703 |
| COMPRESSION INDEX | | SATURATION (%) | 98.818 | 100.000 |
| TYPE SPECIMEN | | VOID RATIO | 1.927 | 1.336 |
| DIA. (in) 2.430 | HT. (in) 0.954 | BACK PRESSURE (t/ft^2) | | |

| | | | | |
|---|---|---|---|---|
| CLASSIFICATION | Brown slightly gray silty Clay with organics | | | |
| LL 0.0 | PL 0.0 | PI 0.0 | PROJECT Lower Guadalupe River | |
| GS 2.720 | $D_{10}$ | | Data File: CB17-2.CON | |
| REMARKS | | | BORING NO. CB-17 | SAMPLE NO. 2 |
| | | | DEPTH 20-22.5 ft | DATE 02/14/02 |

Woodward Clyde Consultants
CONSOLIDATION TEST REPORT

```
Wed Feb 27 11:02:21 2002                                                              Page :  1

                                      CONSOLIDATION TEST DATA

Project : Lower Guadalupe River     Location : San Jose, CA          Project No. : 9500112030
Test No. : CB17-2
Boring No. : CB-17                  Test Date : 02/14/02             Tested by : C. Wason
Sample No. : 2                      Sample Type : Shelby             Depth : 20-22.5 ft
Checked by : S. Capps
Soil Description : Brown slightly gray silty Clay with organics
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

       APPLIED       FINAL       VOID      STRAIN          FITTING              COEFFICIENT OF CONSOLIDATION
      PRESSURE   DISPLACEMENT    RATIO     AT END        TIME (min)                      (in^2/s)
      (t/ft^2)       (in)                   (%)       SQ.RT.       LOG       SQ.RT.        LOG           AVE

  1)    0.06         0.002       1.921      0.21        0.1        0.0     1.01E-002    0.00E+000    0.00E+000
  2)    0.13         0.006       1.910      0.60        0.0        0.0     0.00E+000    0.00E+000    0.00E+000
  3)    0.25         0.018       1.872      1.87       21.3        0.0     3.42E-005    0.00E+000    0.00E+000
  4)    0.50         0.044       1.794      4.56       14.6        0.0     4.79E-005    0.00E+000    0.00E+000
  5)    1.00         0.091       1.646      9.59       16.9        0.0     3.83E-005    0.00E+000    0.00E+000
  6)    2.00         0.155       1.451     16.26       17.9        0.0     3.17E-005    0.00E+000    0.00E+000
  7)    4.00         0.222       1.245     23.29       13.7        0.0     3.50E-005    0.00E+000    0.00E+000
  8)    8.00         0.292       1.030     30.66       20.1        0.0     1.98E-005    0.00E+000    0.00E+000
  9)    1.00         0.258       1.135     27.05        0.0        0.0     0.00E+000    0.00E+000    0.00E+000
 10)    0.06         0.193       1.336     20.21        0.0        0.0     0.00E+000    0.00E+000    0.00E+000
```

## CONSOLIDATION TEST DATA

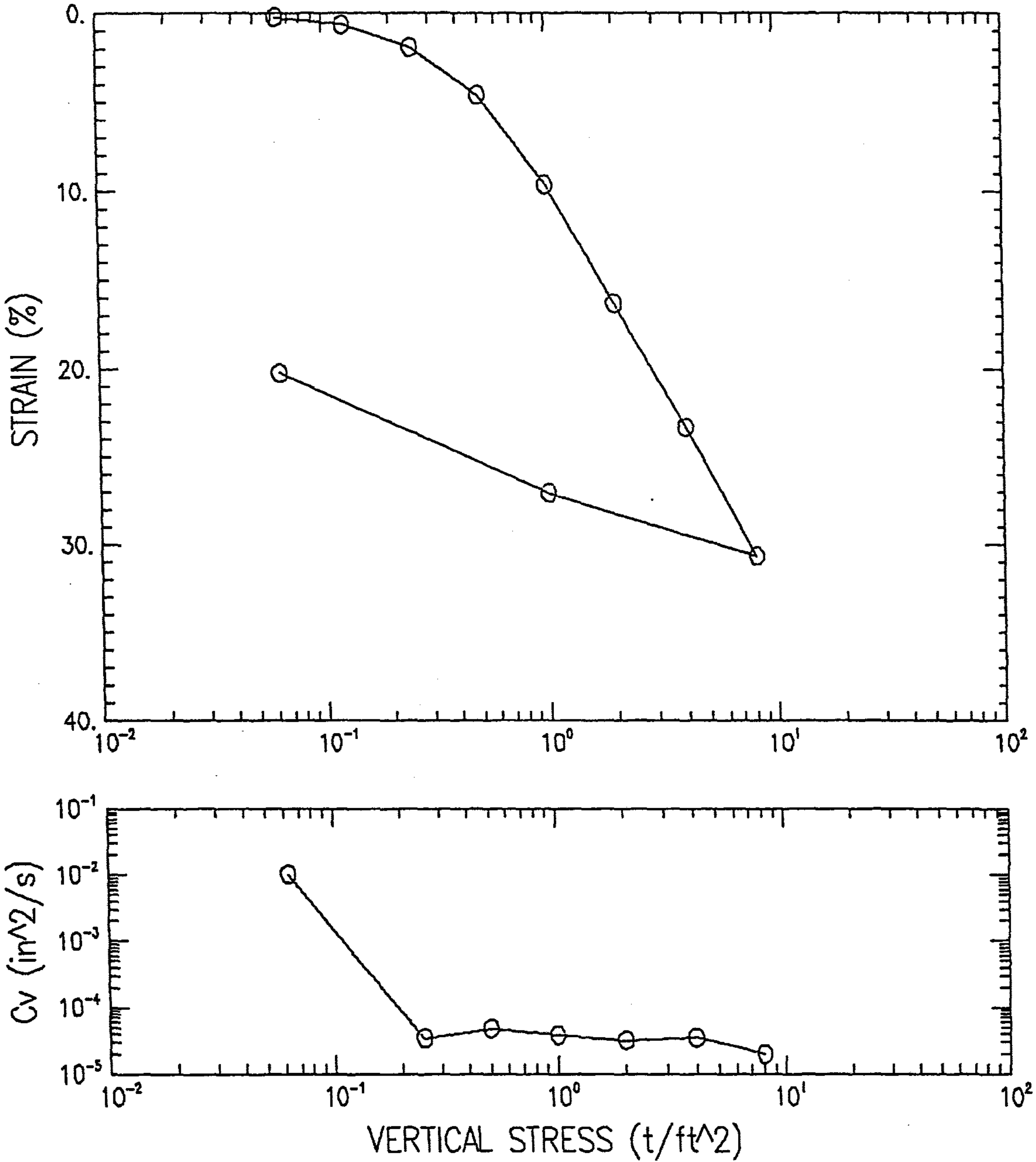
Project : Lower Guadalupe River Location : San Jose, CA Project No. : 9500112030
Test No. : CB17-2
Boring No. : CB-17 Test Date : 02/14/02 Tested by : C. Wason
Sample No. : 2 Sample Type : Shelby Depth : 20-22.5 ft
Checked by : S. Capps
Soil Description : Brown slightly gray silty Clay with organics
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

Liquid Limit : 0 Plastic Limit : 0 Plasticity Index : 0
Specific Gravity : 2.72 Initial Height : 0.95 Sample Diameter : 2.43 (in)
Initial Void Ratio : 1.927 Weight of Piston and top cap : 0.00 (gm)

| | TRIMMINGS | SPECIMEN + RING BEFORE | SPECIMEN + RING AFTER | AFTER |
|---|---|---|---|---|
| CONTAINER NO. | | RING | RING | |
| WT CONTAINER + WET SOIL (gm) | 226.85 | 298.27 | 284.18 | 284.18 |
| WT CONTAINER + DRY SOIL (gm) | 159.02 | 251.10 | 251.10 | 251.10 |
| WT CONTAINER (gm) | 62.15 | 183.73 | 183.73 | 183.73 |
| WT DRY SOIL (gm) | 96.87 | 67.37 | 67.37 | 67.37 |
| WATER CONTENT (%) | 70.02 | 70.02 | 49.10 | 49.10 |
| VOID RATIO | ------ | 1.93 | 1.34 | ------ |
| DEGREE OF SATURATION (%) | ------ | 98.82 | 100.00 | ------ |
| DRY DENSITY (lb/ft^3) | ------ | 58.01 | 72.70 | ------ |

URS

# CONSOLIDATION TEST
# SUMMARY REPORT



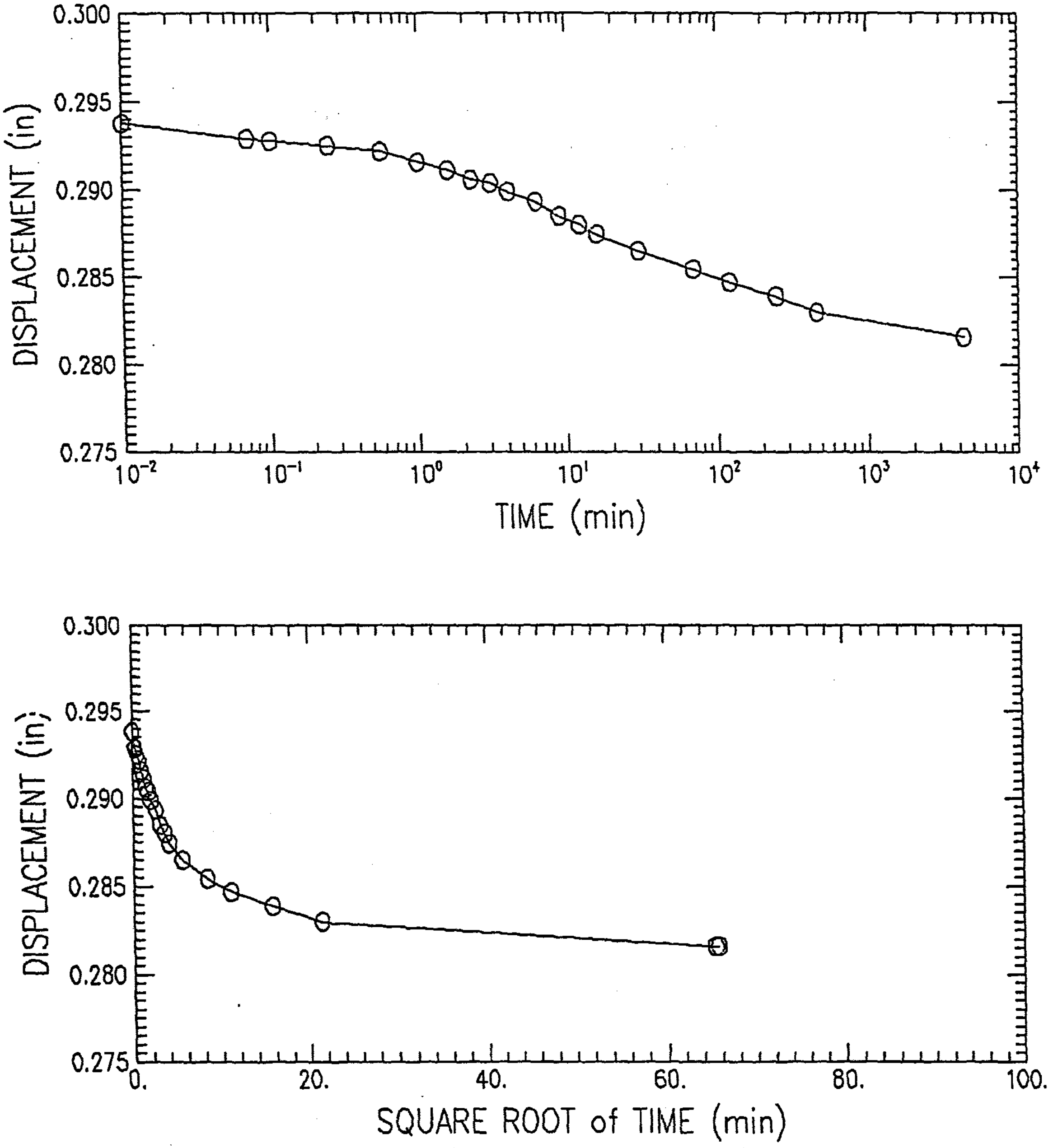
Woodward Clyde Consultants

Project Name : Lower Guadalupe River

| | | |
|---|---|---|
| Project No : 9500112030 | Boring No : CB-17 | Sample No : 2 |
| Test Date : 02/14/02 | Test No : CB17-2 | Depth : 20-22.5 ft |

Description : Brown slightly gray silty Clay with organics

# CONSOLIDATION TEST
# TIME CURVES (STEP 3 OF 10)
# STRESS : 0.25 (t/ft^2)



| | | |
|---|---|---|
| Woodward Clyde Consultants | | |
| Project Name : Lower Guadalupe River | | |
| Project No : 9500112030 | Boring No : CB-17 | Sample No : 2 |
| Test Date : 02/14/02 | Test No : CB17-2 | Depth : 20-22.5 ft |
| Description : Brown slightly gray silty Clay with organics | | |

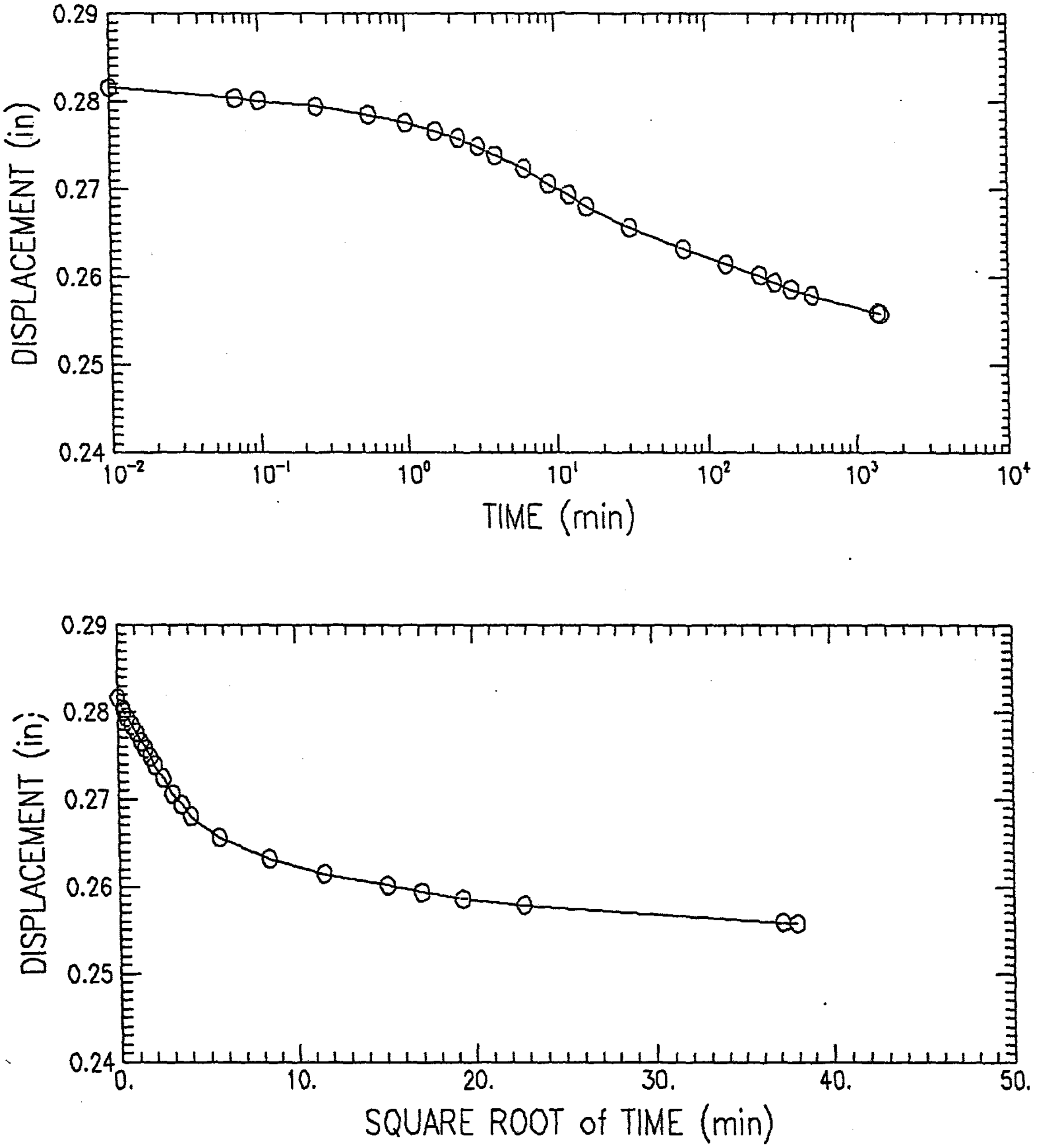
## CONSOLIDATION TEST DATA

Project : Lower Guadalupe River
Location : San Jose, CA
Project No. : 9500112030
Test No. : CB17-2
Boring No. : CB-17
Test Date : 02/14/02
Tested by : C. Wason
Sample No. : 2
Sample Type : Shelby
Depth : 20-22.5 ft
Checked by : S. Capps
Soil Description : Brown slightly gray silty Clay with organics
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

Load Increment : 3/10
Stress increment from 0.13 (t/ft^2) to 0.25 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.915 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0009 | 1.912 | 0.09 |
| 3) | | | 0.10 | 0.32 | 0.0010 | 1.912 | 0.10 |
| 4) | | | 0.25 | 0.50 | 0.0013 | 1.911 | 0.14 |
| 5) | | | 0.57 | 0.75 | 0.0016 | 1.910 | 0.17 |
| 6) | | | 1.00 | 1.00 | 0.0022 | 1.908 | 0.23 |
| 7) | | | 1.57 | 1.25 | 0.0026 | 1.907 | 0.28 |
| 8) | | | 2.25 | 1.50 | 0.0032 | 1.905 | 0.34 |
| 9) | | | 3.07 | 1.75 | 0.0034 | 1.905 | 0.36 |
| 10) | | | 4.00 | 2.00 | 0.0039 | 1.903 | 0.41 |
| 11) | | | 6.25 | 2.50 | 0.0045 | 1.901 | 0.47 |
| 12) | | | 9.00 | 3.00 | 0.0053 | 1.899 | 0.56 |
| 13) | | | 12.25 | 3.50 | 0.0058 | 1.897 | 0.61 |
| 14) | | | 16.00 | 4.00 | 0.0064 | 1.896 | 0.67 |
| 15) | | | 30.00 | 5.48 | 0.0073 | 1.893 | 0.77 |
| 16) | | | 69.00 | 8.31 | 0.0084 | 1.889 | 0.88 |
| 17) | | | 120.00 | 10.95 | 0.0091 | 1.887 | 0.95 |
| 18) | | | 244.00 | 15.62 | 0.0099 | 1.885 | 1.04 |
| 19) | | | 457.00 | 21.38 | 0.0108 | 1.882 | 1.13 |
| 20) | | | 4264.00 | 65.30 | 0.0122 | 1.878 | 1.28 |
| 21) | | | 4309.00 | 65.64 | 0.0122 | 1.878 | 1.28 |

# CONSOLIDATION TEST
## TIME CURVES (STEP 4 OF 10)
## STRESS : 0.5 (t/ft^2)



Woodward Clyde Consultants

Project Name : Lower Guadalupe River

Project No : 9500112030 Boring No : CB-17 Sample No : 2

Test Date : 02/14/02 Test No : CB17-2 Depth : 20-22.5 ft

Description : Brown slightly gray silty Clay with organics

Wed Feb 27 11:10:59 2002

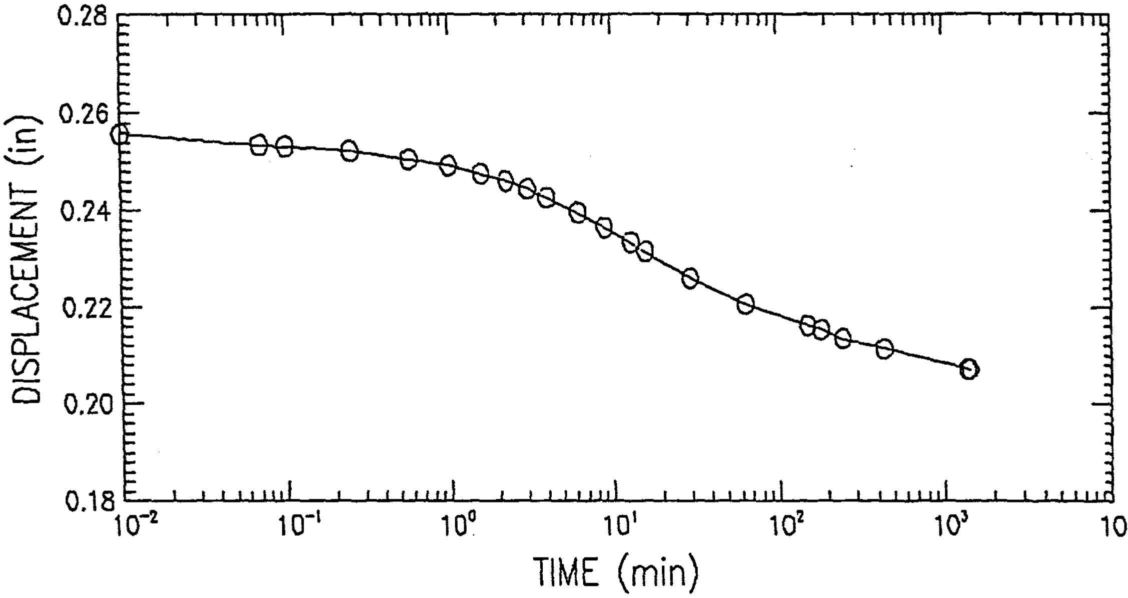
# CONSOLIDATION TEST DATA

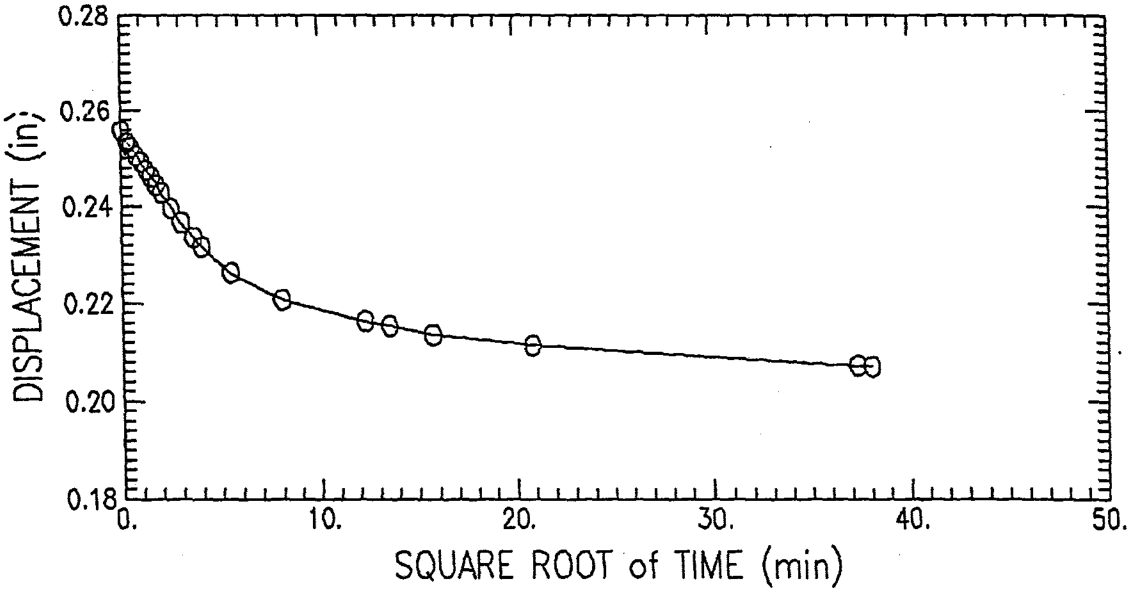
Project : Lower Guadalupe River
Location : San Jose, CA
Project No. : 9500112030
Test No. : CB17-2
Boring No. : CB-17
Test Date : 02/14/02
Tested by : C. Wason
Sample No. : 2
Sample Type : Shelby
Depth : 20-22.5 ft
Checked by : S. Capps
Soil Description : Brown slightly gray silty Clay with organics
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

Load Increment : 4/10
Stress increment from 0.25 (t/ft^2) to 0.50 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.890 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0012 | 1.886 | 0.13 |
| 3) | | | 0.10 | 0.32 | 0.0015 | 1.885 | 0.16 |
| 4) | | | 0.25 | 0.50 | 0.0022 | 1.883 | 0.23 |
| 5) | | | 0.57 | 0.75 | 0.0031 | 1.880 | 0.32 |
| 6) | | | 1.00 | 1.00 | 0.0040 | 1.877 | 0.42 |
| 7) | | | 1.57 | 1.25 | 0.0050 | 1.874 | 0.52 |
| 8) | | | 2.25 | 1.50 | 0.0058 | 1.872 | 0.61 |
| 9) | | | 3.07 | 1.75 | 0.0067 | 1.869 | 0.70 |
| 10) | | | 4.00 | 2.00 | 0.0077 | 1.866 | 0.81 |
| 11) | | | 6.25 | 2.50 | 0.0092 | 1.861 | 0.96 |
| 12) | | | 9.00 | 3.00 | 0.0110 | 1.856 | 1.15 |
| 13) | | | 12.25 | 3.50 | 0.0122 | 1.852 | 1.28 |
| 14) | | | 16.00 | 4.00 | 0.0136 | 1.848 | 1.43 |
| 15) | | | 31.00 | 5.57 | 0.0160 | 1.840 | 1.68 |
| 16) | | | 70.00 | 8.37 | 0.0184 | 1.833 | 1.93 |
| 17) | | | 133.00 | 11.53 | 0.0201 | 1.828 | 2.11 |
| 18) | | | 227.00 | 15.07 | 0.0214 | 1.824 | 2.24 |
| 19) | | | 287.00 | 16.94 | 0.0222 | 1.821 | 2.33 |
| 20) | | | 372.00 | 19.29 | 0.0230 | 1.819 | 2.41 |
| 21) | | | 517.00 | 22.74 | 0.0237 | 1.817 | 2.48 |
| 22) | | | 1381.00 | 37.16 | 0.0257 | 1.811 | 2.69 |
| 23) | | | 1442.00 | 37.97 | 0.0258 | 1.810 | 2.70 |

# CONSOLIDATION TEST
# TIME CURVES (STEP 5 OF 10)
# STRESS : 1 (t/ft^2)





Woodward Clyde Consultants
Project Name : Lower Guadalupe River
Project No : 9500112030 Boring No : CB-17 Sample No : 2
Test Date : 02/14/02 Test No : CB17-2 Depth : 20-22.5 ft
Description : Brown slightly gray silty Clay with organics

Wed Feb 27 11:11:17 2002

# CONSOLIDATION TEST DATA

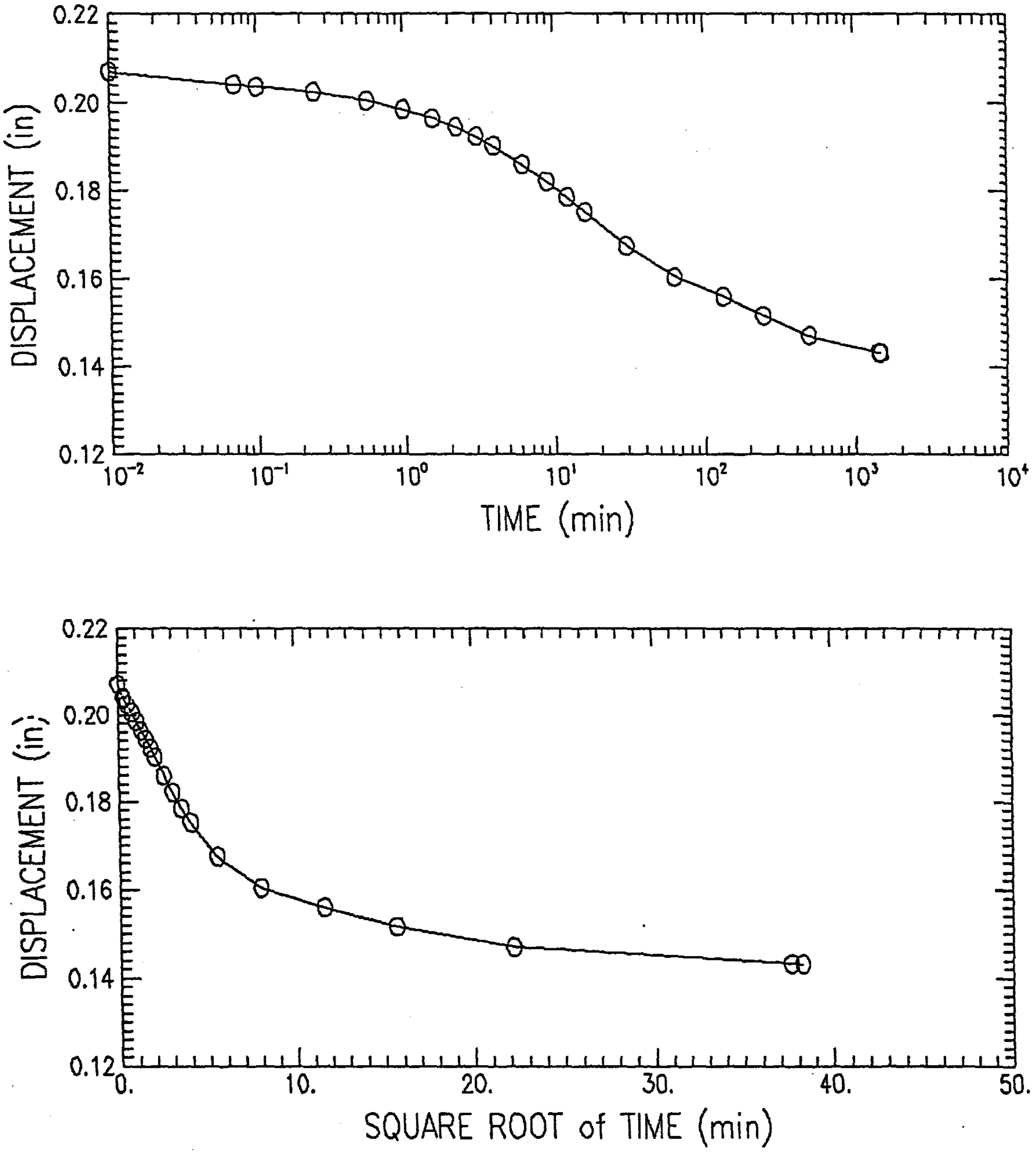
Project : Lower Guadalupe River  Location : San Jose, CA  Project No. : 9500112030
Test No. : CB17-2
Boring No. : CB-17  Test Date : 02/14/02  Tested by : C. Wason
Sample No. : 2  Sample Type : Shelby  Depth : 20-22.5 ft
Checked by : S. Capps
Soil Description : Brown slightly gray silty Clay with organics
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

Load Increment : 5/10
Stress increment from 0.50 (t/ft^2) to 1.00 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.848 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0023 | 1.841 | 0.24 |
| 3) | | | 0.10 | 0.32 | 0.0026 | 1.840 | 0.27 |
| 4) | | | 0.25 | 0.50 | 0.0035 | 1.837 | 0.37 |
| 5) | | | 0.57 | 0.75 | 0.0052 | 1.832 | 0.55 |
| 6) | | | 1.00 | 1.00 | 0.0065 | 1.828 | 0.68 |
| 7) | | | 1.57 | 1.25 | 0.0082 | 1.823 | 0.86 |
| 8) | | | 2.25 | 1.50 | 0.0096 | 1.818 | 1.01 |
| 9) | | | 3.07 | 1.75 | 0.0113 | 1.813 | 1.18 |
| 10) | | | 4.00 | 2.00 | 0.0131 | 1.808 | 1.37 |
| 11) | | | 6.25 | 2.50 | 0.0162 | 1.798 | 1.70 |
| 12) | | | 9.00 | 3.00 | 0.0192 | 1.789 | 2.01 |
| 13) | | | 13.00 | 3.61 | 0.0223 | 1.779 | 2.34 |
| 14) | | | 16.00 | 4.00 | 0.0242 | 1.774 | 2.54 |
| 15) | | | 30.00 | 5.48 | 0.0296 | 1.757 | 3.10 |
| 16) | | | 64.00 | 8.00 | 0.0351 | 1.740 | 3.68 |
| 17) | | | 151.00 | 12.29 | 0.0395 | 1.727 | 4.14 |
| 18) | | | 183.00 | 13.53 | 0.0404 | 1.724 | 4.23 |
| 19) | | | 247.00 | 15.72 | 0.0423 | 1.718 | 4.43 |
| 20) | | | 435.00 | 20.86 | 0.0444 | 1.712 | 4.65 |
| 21) | | | 1391.00 | 37.30 | 0.0486 | 1.699 | 5.09 |
| 22) | | | 1447.00 | 38.04 | 0.0487 | 1.698 | 5.10 |

**URS**

# CONSOLIDATION TEST
# TIME CURVES (STEP 6 OF 10)
# STRESS : 2 (t/ft^2)



Woodward Clyde Consultants

Project Name : Lower Guadalupe River

Project No : 9500112030 Boring No : CB-17 Sample No : 2

Test Date : 02/14/02 Test No : CB17-2 Depth : 20-22.5 ft

Description : Brown slightly gray silty Clay with organics

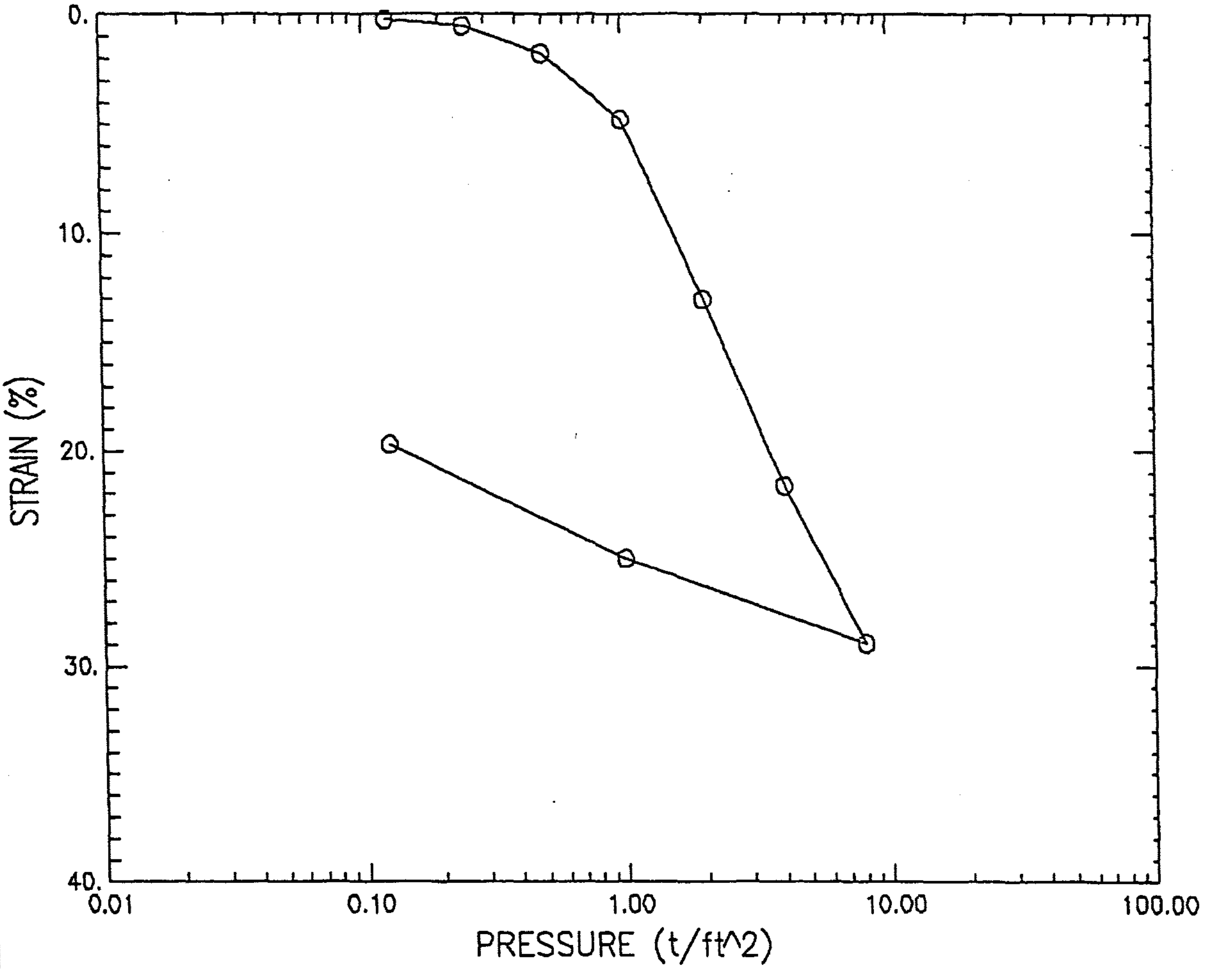
# CONSOLIDATION TEST DATA

Project : Lower Guadalupe River
Location : San Jose, CA
Project No. : 9500112030
Test No. : CB17-2
Boring No. : CB-17
Test Date : 02/14/02
Tested by : C. Wason
Sample No. : 2
Sample Type : Shelby
Depth : 20-22.5 ft
Checked by : S. Capps
Soil Description : Brown slightly gray silty Clay with organics
Remarks : Trimmed from 2.86 down to 2.43 inches in diameter

Load Increment : 6/10
Stress increment from 1.00 (t/ft^2) to 2.00 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.778 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0029 | 1.769 | 0.30 |
| 3) | | | 0.10 | 0.32 | 0.0034 | 1.767 | 0.36 |
| 4) | | | 0.25 | 0.50 | 0.0046 | 1.763 | 0.48 |
| 5) | | | 0.57 | 0.75 | 0.0066 | 1.757 | 0.69 |
| 6) | | | 1.00 | 1.00 | 0.0086 | 1.751 | 0.90 |
| 7) | | | 1.57 | 1.25 | 0.0106 | 1.745 | 1.11 |
| 8) | | | 2.25 | 1.50 | 0.0126 | 1.739 | 1.32 |
| 9) | | | 3.07 | 1.75 | 0.0147 | 1.732 | 1.54 |
| 10) | | | 4.00 | 2.00 | 0.0168 | 1.726 | 1.76 |
| 11) | | | 6.25 | 2.50 | 0.0211 | 1.713 | 2.21 |
| 12) | | | 9.00 | 3.00 | 0.0249 | 1.701 | 2.61 |
| 13) | | | 12.25 | 3.50 | 0.0286 | 1.690 | 3.00 |
| 14) | | | 16.00 | 4.00 | 0.0319 | 1.680 | 3.34 |
| 15) | | | 30.00 | 5.48 | 0.0396 | 1.656 | 4.15 |
| 16) | | | 63.00 | 7.94 | 0.0466 | 1.635 | 4.88 |
| 17) | | | 134.00 | 11.58 | 0.0511 | 1.621 | 5.36 |
| 18) | | | 245.00 | 15.65 | 0.0554 | 1.608 | 5.81 |
| 19) | | | 493.00 | 22.20 | 0.0600 | 1.593 | 6.29 |
| 20) | | | 1417.00 | 37.64 | 0.0638 | 1.582 | 6.69 |
| 21) | | | 1465.00 | 38.28 | 0.0639 | 1.582 | 6.70 |

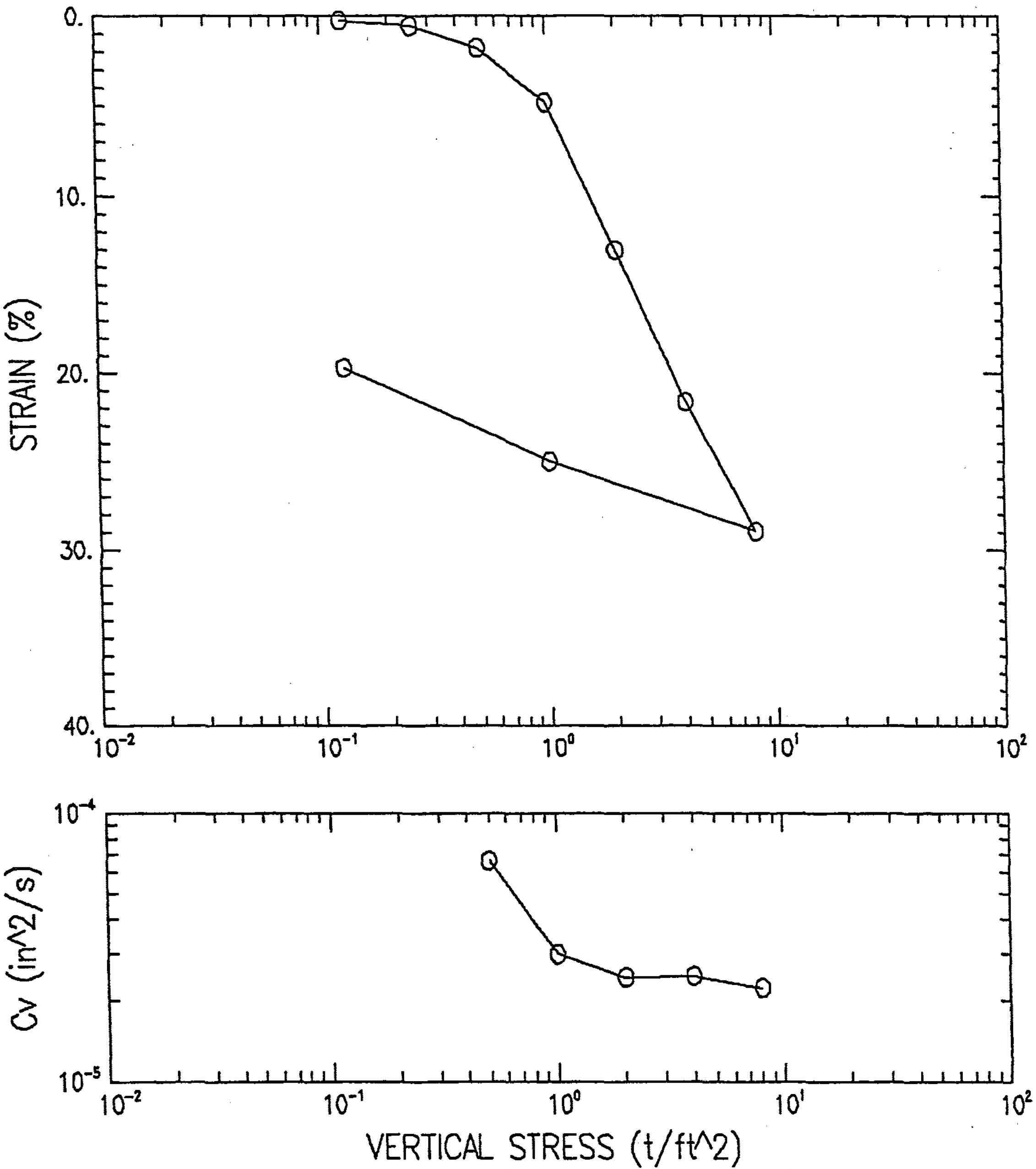
**URS**



| | | | BEFORE TEST | AFTER TEST |
|---|---|---|---|---|
| OVERBURDEN PRESSURE (t/ft^2) | | WATER CONTENT (%) | 66.949 | 47.270 |
| PRECONSOL. PRESSURE (t/ft^2) | | DRY DENSITY (lb/ft^3) | 60.609 | 75.458 |
| COMPRESSION INDEX | | SATURATION (%) | 99.124 | 99.998 |
| TYPE SPECIMEN | | VOID RATIO | 1.905 | 1.333 |
| DIA. (in) 2.425 | HT. (in) 0.950 | BACK PRESSURE (t/ft^2) | | |
| CLASSIFICATION Gray silty Clay | | | | |
| LL 0.0 | PL 0.0 <br> PI 0.0 | PROJECT Lower Guadalupe River | | |
| GS 2.820 | $D_{10}$ | Data File: CB17-4.CON | | |
| REMARKS | | BORING NO. CB-17 | | SAMPLE NO. 4 |
| | | DEPTH 30-32.5 ft | | DATE 02/14/02 |

Woodward Clyde Consultants
CONSOLIDATION TEST REPORT

# CONSOLIDATION TEST
# SUMMARY REPORT



Woodward Clyde Consultants
Project Name : Lower Guadalupe River
Project No : 9500112030 Boring No : CB-17 Sample No : 4
Test Date : 02/14/02 Test No : CB17-4 Depth : 30-32.5 ft
Description : Gray silty Clay

## CONSOLIDATION TEST DATA

Project : Lower Guadalupe River Location : San Jose, CA Project No. : 9500112030
Test No. : CB17-4
Boring No. : CB-17 Test Date : 02/14/02 Tested by : C. Wason
Sample No. : 4 Sample Type : Shelby Depth : 30-32.5 ft
Checked by : S. Capps
Soil Description : Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.425 inches in diameter

| | APPLIED PRESSURE (t/ft^2) | FINAL DISPLACEMENT (in) | VOID RATIO | STRAIN AT END (%) | FITTING TIME (min) | | COEFFICIENT OF CONSOLIDATION (in^2/s) | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | SQ.RT. | LOG | SQ.RT. | LOG | AVE |
| 1) | 0.13 | 0.002 | 1.898 | 0.24 | 0.0 | 0.0 | 0.00E+000 | 0.00E+000 | 0.00E+000 |
| 2) | 0.25 | 0.005 | 1.889 | 0.53 | 0.0 | 0.0 | 0.00E+000 | 0.00E+000 | 0.00E+000 |
| 3) | 0.50 | 0.017 | 1.854 | 1.76 | 10.9 | 0.0 | 6.60E-005 | 0.00E+000 | 0.00E+000 |
| 4) | 1.00 | 0.045 | 1.766 | 4.78 | 23.2 | 0.0 | 2.98E-005 | 0.00E+000 | 0.00E+000 |
| 5) | 2.00 | 0.124 | 1.527 | 13.01 | 25.2 | 0.0 | 2.43E-005 | 0.00E+000 | 0.00E+000 |
| 6) | 4.00 | 0.205 | 1.276 | 21.63 | 20.5 | 0.0 | 2.47E-005 | 0.00E+000 | 0.00E+000 |
| 7) | 8.00 | 0.275 | 1.064 | 28.95 | 18.6 | 0.0 | 2.22E-005 | 0.00E+000 | 0.00E+000 |
| 8) | 1.00 | 0.237 | 1.178 | 25.00 | 0.0 | 0.0 | 0.00E+000 | 0.00E+000 | 0.00E+000 |
| 9) | 0.13 | 0.187 | 1.333 | 19.69 | 0.0 | 0.0 | 0.00E+000 | 0.00E+000 | 0.00E+000 |

## CONSOLIDATION TEST DATA

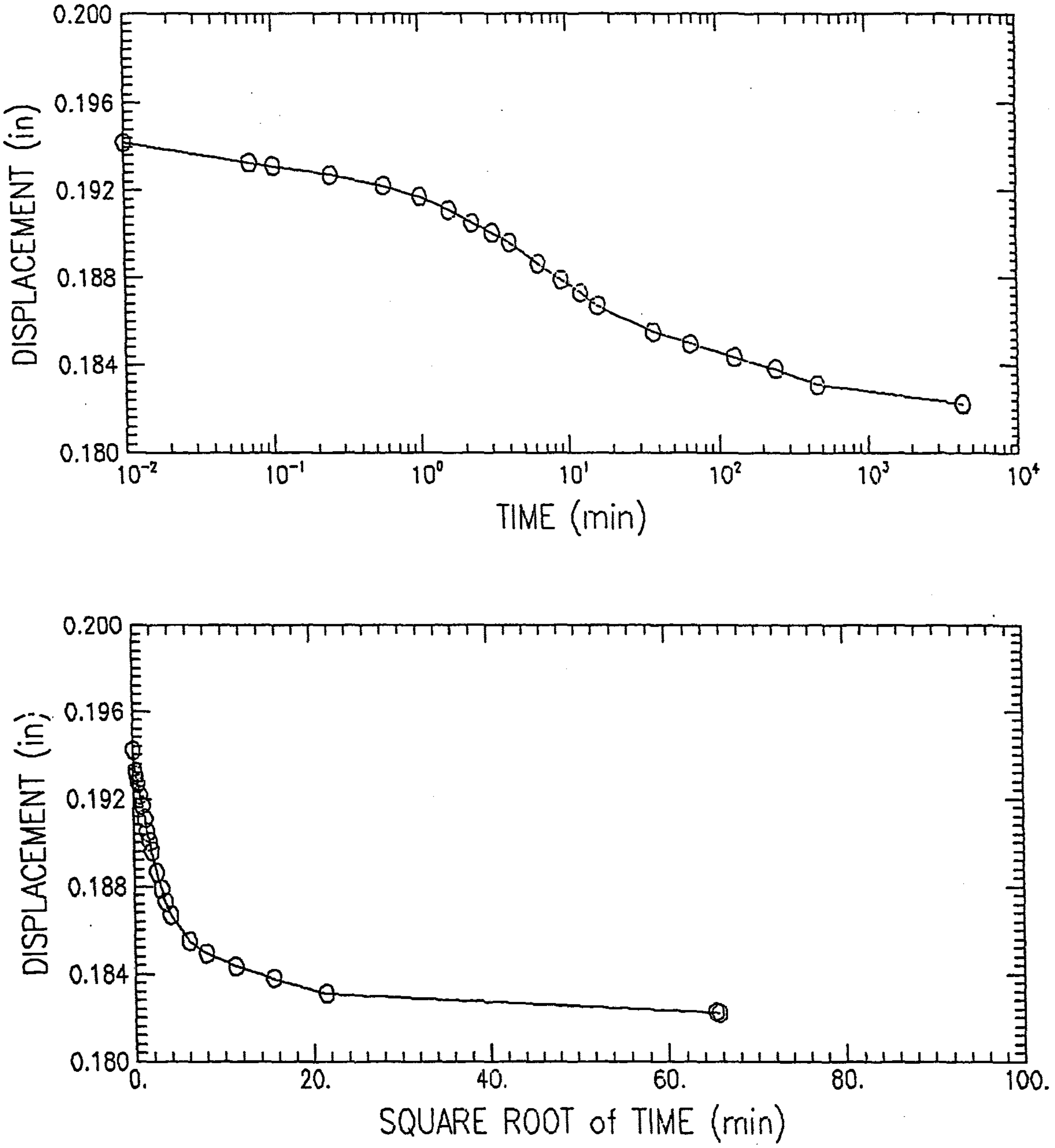
Project : Lower Guadalupe River
Location : San Jose, CA
Project No. : 9500112030
Test No. : CB17-4
Boring No. : CB-17
Test Date : 02/14/02
Tested by : C. Wason
Sample No. : 4
Sample Type : Shelby
Depth : 30-32.5 ft
Checked by : S. Capps
Soil Description : Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.425 inches in diameter

Liquid Limit : 0
Plastic Limit : 0
Plasticity Index : 0
Specific Gravity : 2.82
Initial Height : 0.95
Sample Diameter : 2.43 (in)
Initial Void Ratio : 1.905
Weight of Piston and top cap : 0.00 (gm)

| | TRIMMINGS | SPECIMEN + RING BEFORE | SPECIMEN + RING AFTER | AFTER |
|---|---|---|---|---|
| CONTAINER NO. | | RING | RING | |
| WT CONTAINER + WET SOIL (gm) | 215.55 | 304.21 | 290.48 | 290.48 |
| WT CONTAINER + DRY SOIL (gm) | 154.11 | 257.50 | 257.50 | 257.50 |
| WT CONTAINER (gm) | 62.36 | 187.73 | 187.73 | 187.73 |
| WT DRY SOIL (gm) | 91.75 | 69.77 | 69.77 | 69.77 |
| WATER CONTENT (%) | 66.96 | 66.95 | 47.27 | 47.27 |
| VOID RATIO | ------ | 1.90 | 1.33 | ------ |
| DEGREE OF SATURATION (%) | ------ | 99.12 | 100.00 | ------ |
| DRY DENSITY (lb/ft^3) | ------ | 60.61 | 75.46 | ------ |

**URS**

# CONSOLIDATION TEST
## TIME CURVES (STEP 3 OF 9)
## STRESS : 0.5 (t/ft^2)



Woodward Clyde Consultants
Project Name : Lower Guadalupe River
Project No : 9500112030 Boring No : CB-17 Sample No : 4
Test Date : 02/14/02 Test No : CB17-4 Depth : 30-32.5 ft
Description : Gray silty Clay

Tue Feb 26 11:37:14 2002

# CONSOLIDATION TEST DATA

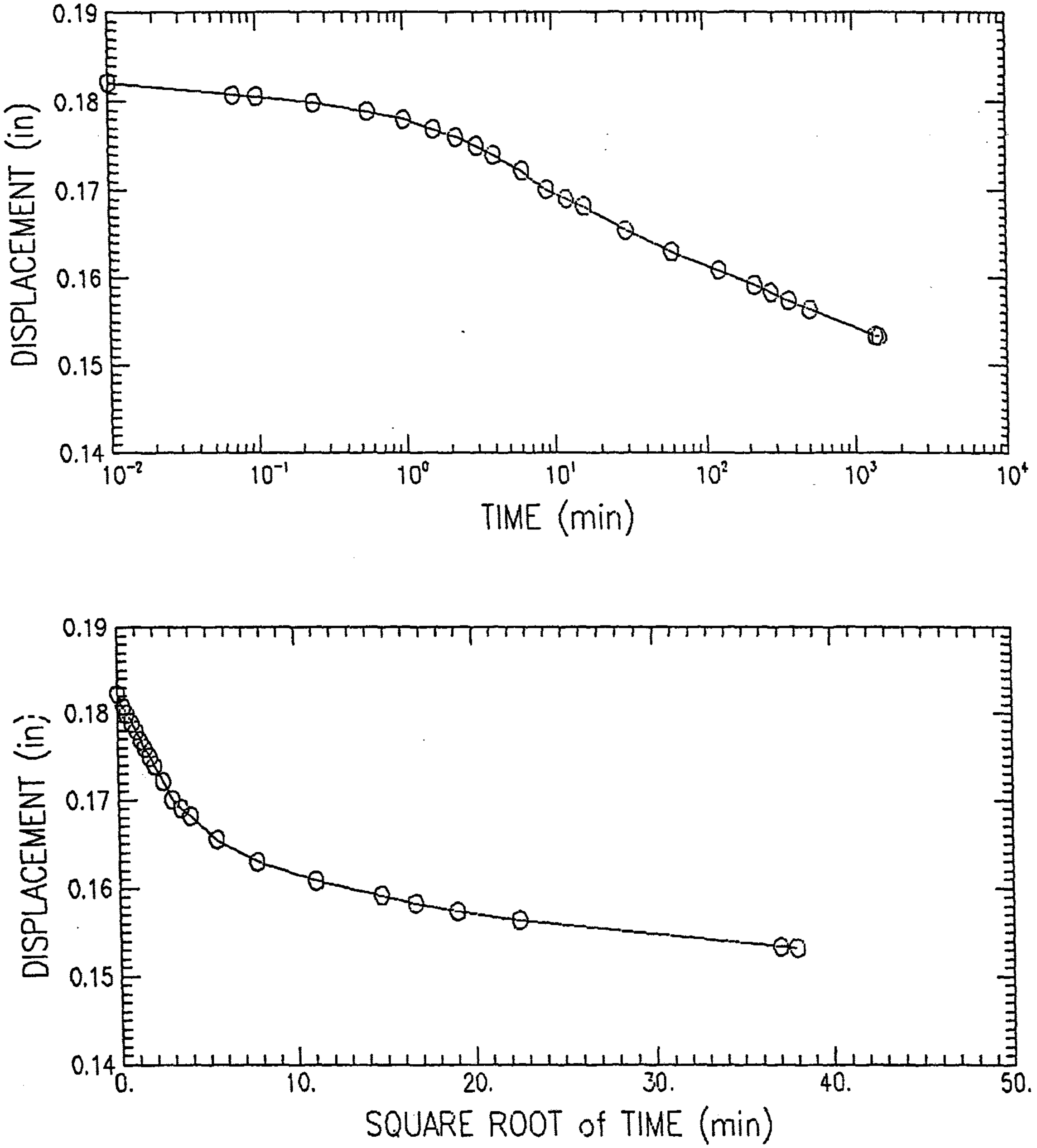
Project : Lower Guadalupe River
Location : San Jose, CA
Project No. : 9500112030
Test No. : CB17-4
Boring No. : CB-17
Test Date : 02/14/02
Tested by : C. Wason
Sample No. : 4
Sample Type : Shelby
Depth : 30-32.5 ft
Checked by : S. Capps
Soil Description : Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.425 inches in diameter

Load Increment : 3/9
Stress increment from 0.25 (t/ft^2) to 0.50 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.896 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0010 | 1.893 | 0.10 |
| 3) | | | 0.10 | 0.32 | 0.0011 | 1.892 | 0.12 |
| 4) | | | 0.25 | 0.50 | 0.0015 | 1.891 | 0.16 |
| 5) | | | 0.57 | 0.75 | 0.0020 | 1.889 | 0.21 |
| 6) | | | 1.00 | 1.00 | 0.0025 | 1.888 | 0.26 |
| 7) | | | 1.57 | 1.25 | 0.0031 | 1.886 | 0.33 |
| 8) | | | 2.25 | 1.50 | 0.0037 | 1.884 | 0.39 |
| 9) | | | 3.07 | 1.75 | 0.0042 | 1.883 | 0.44 |
| 10) | | | 4.00 | 2.00 | 0.0046 | 1.881 | 0.48 |
| 11) | | | 6.25 | 2.50 | 0.0055 | 1.879 | 0.58 |
| 12) | | | 9.00 | 3.00 | 0.0063 | 1.876 | 0.66 |
| 13) | | | 12.25 | 3.50 | 0.0069 | 1.874 | 0.73 |
| 14) | | | 16.00 | 4.00 | 0.0075 | 1.873 | 0.79 |
| 15) | | | 37.00 | 6.08 | 0.0087 | 1.869 | 0.92 |
| 16) | | | 64.00 | 8.00 | 0.0093 | 1.867 | 0.97 |
| 17) | | | 127.00 | 11.27 | 0.0098 | 1.865 | 1.04 |
| 18) | | | 240.00 | 15.49 | 0.0104 | 1.864 | 1.10 |
| 19) | | | 462.00 | 21.49 | 0.0111 | 1.862 | 1.17 |
| 20) | | | 4260.00 | 65.27 | 0.0120 | 1.859 | 1.26 |
| 21) | | | 4314.00 | 65.68 | 0.0120 | 1.859 | 1.26 |

**URS**

# CONSOLIDATION TEST
# TIME CURVES (STEP 4 OF 9)
# STRESS : 1 (t/ft^2)



Woodward Clyde Consultants
Project Name : Lower Guadalupe River
Project No : 9500112030 Boring No : CB-17 Sample No : 4
Test Date : 02/14/02 Test No : CB17-4 Depth : 30-32.5 ft
Description : Gray silty Clay

Tue Feb 26 11:37:36 2002

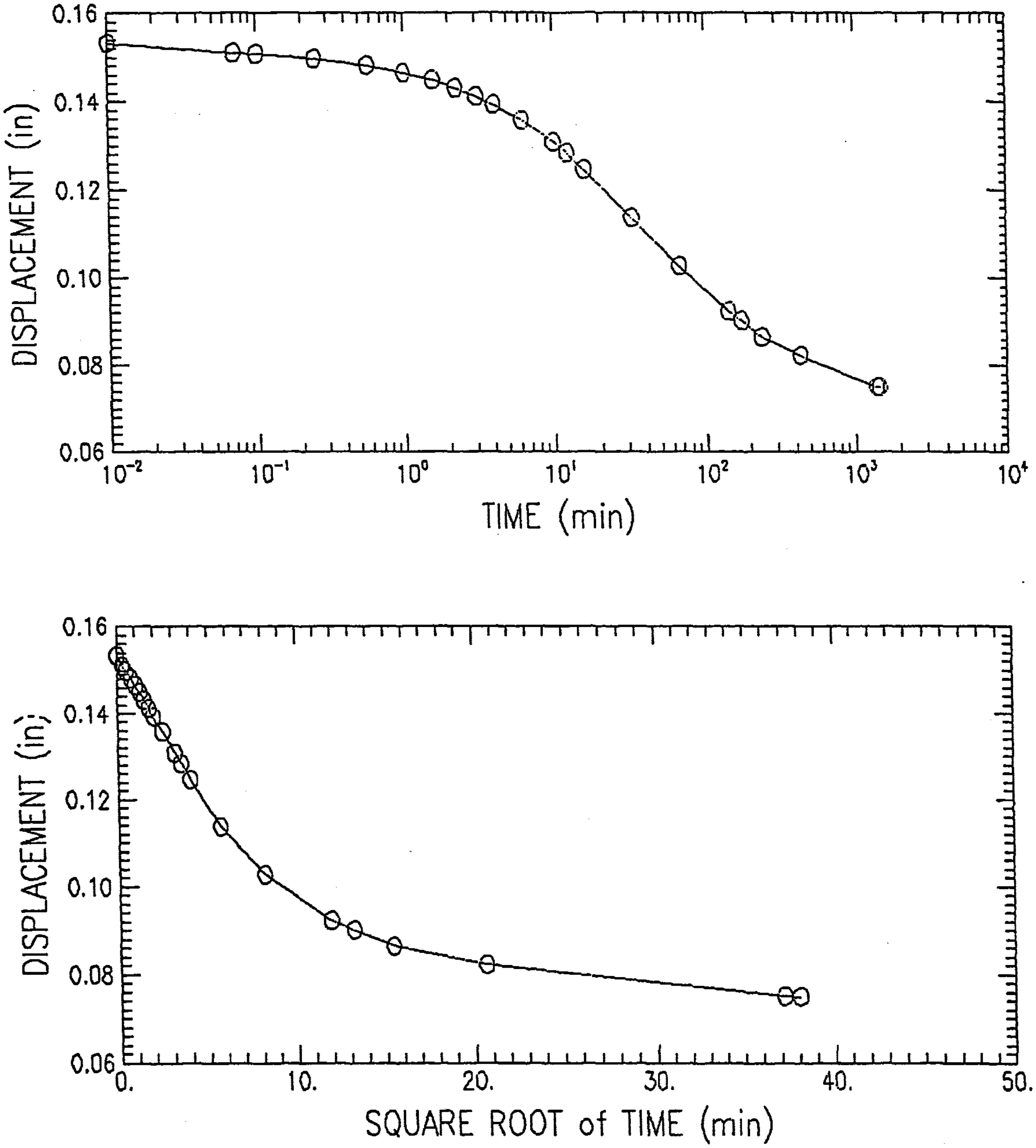
## CONSOLIDATION TEST DATA

Project : Lower Guadalupe River
Location : San Jose, CA
Project No. : 9500112030
Test No. : CB17-4
Boring No. : CB-17
Test Date : 02/14/02
Tested by : C. Wason
Sample No. : 4
Sample Type : Shelby
Depth : 30-32.5 ft
Checked by : S. Capps
Soil Description : Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.425 inches in diameter

Load Increment : 4/9
Stress increment from 0.50 (t/ft^2) to 1.00 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.868 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0014 | 1.864 | 0.15 |
| 3) | | | 0.10 | 0.32 | 0.0016 | 1.863 | 0.17 |
| 4) | | | 0.25 | 0.50 | 0.0023 | 1.861 | 0.24 |
| 5) | | | 0.57 | 0.75 | 0.0033 | 1.858 | 0.35 |
| 6) | | | 1.00 | 1.00 | 0.0042 | 1.855 | 0.44 |
| 7) | | | 1.57 | 1.25 | 0.0053 | 1.852 | 0.56 |
| 8) | | | 2.25 | 1.50 | 0.0062 | 1.849 | 0.65 |
| 9) | | | 3.07 | 1.75 | 0.0072 | 1.846 | 0.76 |
| 10) | | | 4.00 | 2.00 | 0.0082 | 1.843 | 0.86 |
| 11) | | | 6.25 | 2.50 | 0.0100 | 1.838 | 1.05 |
| 12) | | | 9.00 | 3.00 | 0.0121 | 1.831 | 1.27 |
| 13) | | | 12.25 | 3.50 | 0.0131 | 1.828 | 1.38 |
| 14) | | | 16.00 | 4.00 | 0.0140 | 1.825 | 1.47 |
| 15) | | | 30.00 | 5.48 | 0.0167 | 1.817 | 1.76 |
| 16) | | | 60.00 | 7.75 | 0.0192 | 1.810 | 2.02 |
| 17) | | | 123.00 | 11.09 | 0.0213 | 1.803 | 2.24 |
| 18) | | | 217.00 | 14.73 | 0.0230 | 1.798 | 2.42 |
| 19) | | | 277.00 | 16.64 | 0.0239 | 1.795 | 2.52 |
| 20) | | | 362.00 | 19.03 | 0.0248 | 1.792 | 2.61 |
| 21) | | | 507.00 | 22.52 | 0.0258 | 1.789 | 2.72 |
| 22) | | | 1371.00 | 37.03 | 0.0288 | 1.780 | 3.03 |
| 23) | | | 1442.00 | 37.97 | 0.0289 | 1.780 | 3.04 |

# CONSOLIDATION TEST
# TIME CURVES (STEP 5 OF 9)
# STRESS : 2 (t/ft^2)



Woodward Clyde Consultants
Project Name : Lower Guadalupe River
Project No : 9500112030 Boring No : CB-17 Sample No : 4
Test Date : 02/14/02 Test No : CB17-4 Depth : 30-32.5 ft
Description : Gray silty Clay

Tue Feb 26 11:37:55 2002

## CONSOLIDATION TEST DATA

Project : Lower Guadalupe River
Location : San Jose, CA
Project No. : 9500112030
Test No. : CB17-4
Boring No. : CB-17
Test Date : 02/14/02
Tested by : C. Wason
Sample No. : 4
Sample Type : Shelby
Depth : 30-32.5 ft
Checked by : S. Capps
Soil Description : Gray silty Clay
Remarks : Trimmed from 2.86 down to 2.425 inches in diameter

Load Increment : 5/9
Stress increment from 1.00 (t/ft^2) to 2.00 (t/ft^2)

| | DATE | TIME | ELAPSED TIME (min) | SQRT. OF TIME (min) | CHANGE IN HEIGHT (in) | VOID RATIO | STRAIN (%) |
|---|---|---|---|---|---|---|---|
| 1) | | | 0.00 | 0.00 | 0.0000 | 1.817 | 0.00 |
| 2) | | | 0.07 | 0.26 | 0.0022 | 1.810 | 0.23 |
| 3) | | | 0.10 | 0.32 | 0.0025 | 1.809 | 0.26 |
| 4) | | | 0.25 | 0.50 | 0.0035 | 1.806 | 0.37 |
| 5) | | | 0.57 | 0.75 | 0.0051 | 1.801 | 0.54 |
| 6) | | | 1.00 | 1.00 | 0.0068 | 1.796 | 0.72 |
| 7) | | | 1.57 | 1.25 | 0.0084 | 1.791 | 0.88 |
| 8) | | | 2.25 | 1.50 | 0.0102 | 1.785 | 1.07 |
| 9) | | | 3.07 | 1.75 | 0.0121 | 1.780 | 1.27 |
| 10) | | | 4.00 | 2.00 | 0.0140 | 1.774 | 1.47 |
| 11) | | | 6.25 | 2.50 | 0.0175 | 1.763 | 1.84 |
| 12) | | | 10.00 | 3.16 | 0.0224 | 1.748 | 2.36 |
| 13) | | | 12.25 | 3.50 | 0.0249 | 1.740 | 2.62 |
| 14) | | | 16.00 | 4.00 | 0.0286 | 1.729 | 3.01 |
| 15) | | | 32.00 | 5.66 | 0.0395 | 1.696 | 4.16 |
| 16) | | | 66.00 | 8.12 | 0.0505 | 1.662 | 5.32 |
| 17) | | | 142.00 | 11.92 | 0.0609 | 1.630 | 6.42 |
| 18) | | | 174.00 | 13.19 | 0.0631 | 1.624 | 6.65 |
| 19) | | | 237.00 | 15.39 | 0.0668 | 1.612 | 7.04 |
| 20) | | | 425.00 | 20.62 | 0.0710 | 1.599 | 7.48 |
| 21) | | | 1381.00 | 37.16 | 0.0782 | 1.577 | 8.24 |
| 22) | | | 1447.00 | 38.04 | 0.0783 | 1.577 | 8.25 |

# APPENDIX C

*URS Greiner Woodward Clyde*
*A Division of URS Corporation*

55 South Market Street, Suite 1650
San Jose, CA 95113
Tel: 408.297.9585
Fax: 408.297.6962
*Offices Worldwide*

April 14, 2000
95-973017NB.00

Mr. Mohammad Khan
Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118-3686

Subject: Preliminary Geotechnical Parameters
Phase I Flood Protection Alternatives Analysis
Lower Guadalupe River Flood Protection Project
Santa Clara County, California

Dear Mr. Khan:

As requested by Mr. John Rogers of CH2MHill, we have developed preliminary geotechnical parameters for use by their Structural Engineer in evaluating the viability of various flood protection alternatives along the project alignment. Mr. Rogers forwarded a list of specific questions. Those questions and our responses follow.

Additionally, Mr. Rogers asked about the potential for future settlement along the alignment. Apparently, the District provided data that suggests as much as 1 foot of settlement has occurred since 1983. A brief discussion of settlement also is included.

## RESPONSES TO CH2MHILL QUESTIONS

### *What are the lateral soil pressure parameters (gamma & phi)?*

Estimated values of gamma and phi are summarized in Table 1, corresponding to the general soil conditions we encountered throughout the various project reaches.

### *What is the soil bearing capacity, say 5' from the top of the levees?*

Estimates of bearing capacity within the existing levee fill are provided in Table 1 for dead plus live load, and for all loads including seismic. As above, the bearing pressure values correspond to the conditions encountered within the corresponding alignment reaches.

### *What is the bearing capacity at the toe of the levee?*

Table 1 also provides preliminary estimates of the bearing capacity at the inboard toe of the levees.

### *What type of piling would be feasible to drive through the levee (concrete, steel H-section or pipe pile)?*

In recent years, square precast prestressed concrete piles generally have been the most economical piles to drive in the Bay Area. Typically, 12-inch and 14-inch sizes have been used to support multi-story buildings and highway bridges. Based on the compositions of the levees (i.e., predominantly clay soils), we anticipate it would be feasible to install precast prestressed concrete piles through the levees and into the underlying native alluvial soils. It is conceivable some localized obstructions could be encountered. However, predrilling could be used to minimize potential problems during driving.

Steel H-pile and pipe pile alternatives also would be feasible, but could be more costly to install. Another potential pile alternative would be cast-in-place concrete piers, also referred to as cast-in-drilled-hole piles (CIDH). We would envision the minimum diameter of CIDH piles might be on the order of 16 to 18 inches. Larger sizes (e.g., 30 to 36 inches) could be used to minimize potential caving problems associated with drilling below groundwater. Casing or the use of drilling mud also could be used to facilitate CIDH pile construction below groundwater.

### *What would be the length of piles for 45 tons of bearing capacity?*

The conditions encountered along most of the project alignment suggest piles would derive capacity through skin friction rather than end bearing. Nonetheless, a dead plus live load design capacity of 45 tons could be achieved with pile embedment lengths in the range of about 40 to 50 feet for piles with a minimum diameter / width of about 12 to 14 inches. For preliminary purposes, we suggest a dead plus live load unit skin friction value of 500 psf be assumed when estimating the embedment length that might be required for either driven piles or CIDH piles. This value can be assumed for the levee fill and underlying native alluvium. A one-third increase can be applied for seismic loading conditions.

### *What is the seismic rock acceleration?*

Based on a seismic review of the project vicinity, we expect the controlling seismic source to be a Mw 8 earthquake along the 1906 segment of the San Andreas fault. This event has the potential of generating median peak horizontal ground accelerations of 0.31 to 0.33 g. However, please refer to Section 2, Site Geology and Seismicity, of our July 14, 1999 draft geotechnical investigation report for a more comprehensive discussion of the seismicity.

### *What is the depth to bedrock or rock-like materials?*

The depth to bedrock is estimated to be in excess of 93 m (300 ft) for the general project area. Please refer to Section 2, Site Geology and Seismicity, of our July 14, 1999 draft geotechnical investigation report.

## ESTIMATED FUTURE LEVEE SETTLEMENT

For preliminary purposes, we have assumed that future raising of the levees would be limited to approximately 0.9 meter (3 feet) for all seven levee sections discussed in our draft geotechnical report. Our estimates are presented below:

| Section | Incremental Estimated Future Settlement Due to 0.9 meter (3 feet) of New Fill | |
|---|---|---|
| | mm | inches |
| 1 | 203 | 8 |
| 2 | 171 | 6¾ |
| 3 | 267 | 10½ |

| | | |
|---|---|---|
| 4 | 38 | 1½ |
| 5 | 235 | 9¼ |
| 6 | 6 | ¼ |
| 7 | 19 | ¾ |

Therefore the future estimated settlement due to 0.9 meter (3 feet) of new fill placed on the levees generally north of Montague Expressway will be of the order of 254 mm (10 inches) and south of Montague Expressway will be of the order 19 mm (3/4 inch). However the transition zone between these two areas is gradual and could be of the order of 1,000 meters or more. The reasons for the wide variation in estimated settlement are: (1) highly compressible clay such as Bay mud or fat clay is present in the Alviso area, (2) some clays are overconsolidated by a ratio of 2 or 3 or greater, and (3) thickness of clay layers is highly variable along the project alignment.

## LIMITATIONS

The geotechnical opinions and parameters presented herein are preliminary and intended to assist CH2MHill in the evaluation of feasible flood protection alternatives. They have been developed with the standard of care of the geotechnical engineering profession. No other warranties are included, either express or implied, as to the professional advice presented. More detailed geotechnical analysis should be conducted as part of the final design.

If any questions should arise, please contact our office.

Sincerely,

Paul J. Boddie

Paul J. Boddie, G.E.
Geotechnical Department Manager

Attachment

Cc: Mr. John Rogers, CH2MHill

### Table 1
### Summary of Preliminary Geotechnical Design Parameters
### Lower Guadalupe River Flood Control Improvement Project
### Santa Clara County, California

| Reach | Moist Unit Weight (pcf) | Internal Friction Angle[1] (φ') | Bearing Capacity at 5 feet below Crest of Levee (psf) | | Bearing Capacity at Toe of Levee (psf) | | Design Unit Skin Friction for Piles (psf) |
|---|---|---|---|---|---|---|---|
| | | | DL+LL | All Loads | DL+LL | All Loads | |
| A | 120 | 29 | 3000 | 4500 | 2500 | 4000 | 500 |
| B | 120 | 29 | 3000 | 4500 | 2500 | 4000 | 500 |
| C | 128 | 28 | 3000 | 4500 | 2000 | 3000 | 500 |
| D | 125 | 30 | 2500 | 4000 | 1500 | 2500 | 500 |
| E | 125 | 30 | 2000 | 3000 | 1500 | 2500 | 500 |
| F | 130 | 31 | 3000 | 4500 | 1500 | 2500 | 500 |
| G | 130 | 28 | 2000 | 3000 | 1000 | 1500 | 500 |

Notes:

(1) Drained, i.e. effective strength parameter.

**URS**

October 23, 2001
95-00112030.00

Mr. David Von Rueden
CH2MHILL
1737 North First Street, Suite 300
San Jose, California 95112

Subject: Supplementary Geotechnical Parameters
Floodwall Design
Lower Guadalupe River
Santa Clara County, California

Dear Mr. Von Rueden:

As requested by Mr. Majdi Kanaan of CH2MHILL in the attached e-mail dated October 16, 2001, we have developed supplemental geotechnical parameters for the design of the levee floodwalls. Mr. Kanaan forwarded a list of specific questions. Those questions and our responses follow. Previously, we provided preliminary geotechnical parameters for levee floodwalls in a letter dated April 14, 2000 to the Santa Clara Valley Water District (Project Number 95-973017NB.00).

## RESPONSES TO CH2MHILL QUESTIONS

### *What are the earth pressure coefficients and wall friction values to be used for design of the CIDH piles?*

The design geotechnical parameters for the design of the CIDH piles are presented in the table below for reaches F and G, as defined by CH2MHILL. The values of unit weight and friction angle are consistent with those provided in our letter dated April 14, 2000. The active and passive earth pressure coefficients provided herein do not include the effect of the wall friction. A schematic of the lateral pressure distribution is presented in Figure 1.

| | Total Unit Weight (pcf) | Submerged Unit Weight (pcf) | Friction (1) Angle (degrees) | Wall Friction Angle | Isolation Factor | $K_A$ | $K_P$ |
|---|---|---|---|---|---|---|---|
| Reach F | 130 | 67.6 | 31 | 20 | 1.0 | 0.32 | 2.0 |
| Reach G | '130 | 67.6 | 28 | 18 | 1.0 | 0.36 | 1.8 |

(1) Drained, i.e. effective strength parameter.

### *What is the uplift on an L-shaped spread footing foundation?*

Assuming a 7.0-foot spread footing width and maximum height of water above the levee of approximately 3.0 feet, the design uplift pressure has a trapezoidal distribution, with maximum and minimum values of 440 pounds per square foot (psf) and 125 psf, respectively. The uplift pressure distribution is presented in Figure 2.

## LIMITATIONS

The geotechnical opinions and parameters presented herein are intended to assist CH2MHILL in the design of floodwall foundations for the Lower Guadalupe River Flood Control Project. They have been developed with the standard of care of the geotechnical engineering profession. No other warranties are included, either express or implied, as to the professional advice presented.

If any questions should arise, please contact our office.

Sincerely,

Paul J. Boddie

Paul J. Boddie, G.E.
Geotechnical Project Manager

Attachments

cc: Mr. Majdi Kanaan, CH2MHILL

**URS**

Page 1 of 1

Job LOWER GUADALUPE RIVER

Project No.

Sheet ____ of ____

Description LATERAL PRESSURE DESIGN PARAMETERS

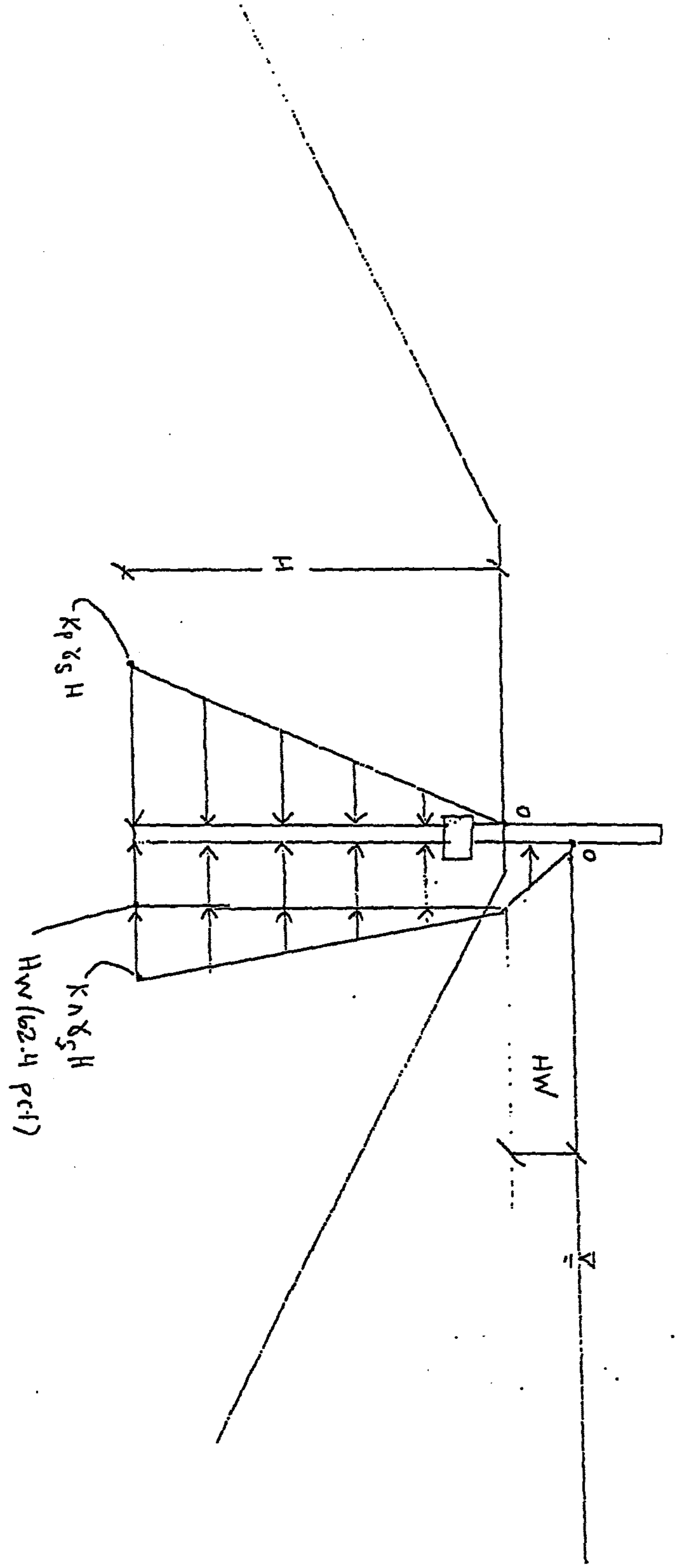
Computed by J. WEELE

Date 10/19/01

Checked by

Date

Reference



$\gamma_s$ = Submerged unit weight = 67.6 pcf.

Figure 1

**URS**

Page 1 of 1

Job LOWER GUADALUPE RIVER

Project No.

Sheet of

Description UPLIFT on 7' shallow foundation

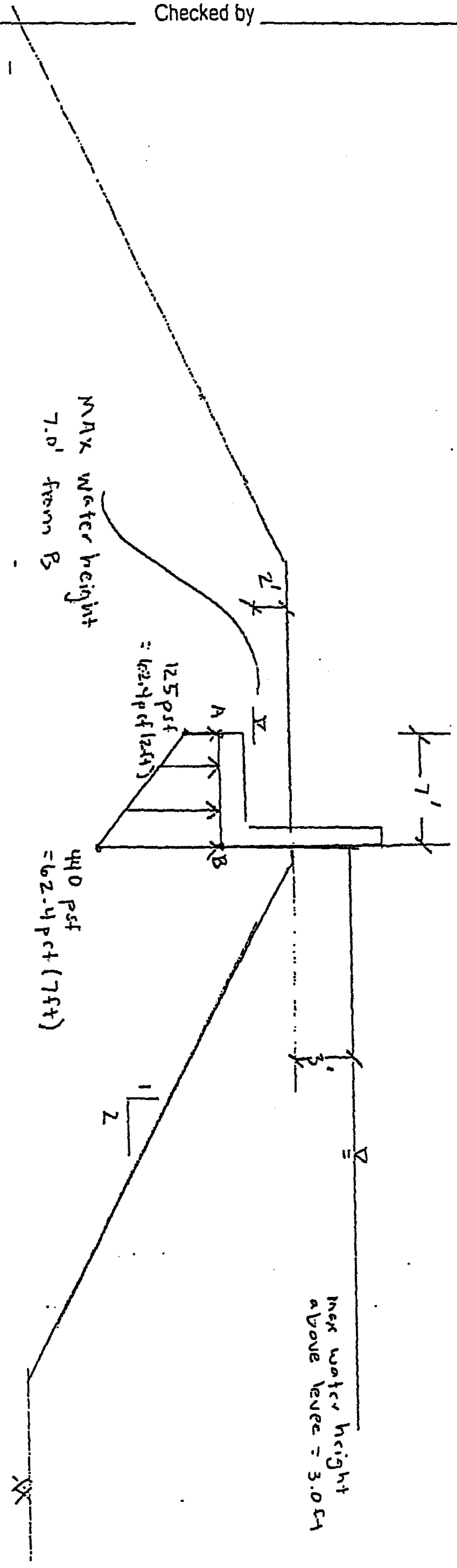
Computed by J.Warle

Date 10/19/01

Checked by

Date

Reference



MAX water height 7.0' from B
2'
125 psf = 62.4pcf(2ft)
A
B
440 psf = 62.4pcf(7ft)
7'
3'
1
2
max water height above levee = 3.0ft

SCALE 1" = 10'

Figure 2

"Kanaan, Majdi/SAC"
<mkanaan@CH2M.com>

10/16/01 11:36 AM

To: Michael_larson@urscorp.com
cc: stephen_huang@urscorp.com, paul_boddie@urscorp.com, "Von Rueden, Dave/SJC" <DRueden@CH2M.com>, "Tamannaie, Nader/SAC" <NTamanna@CH2M.com>, "Gomez, Erika/SJC" <egomez@CH2M.com>
Subject: Lower Guadalupe River project

<<29169.tif>>
As a follow up on our conversation today, I need your recommendations at each reach and at Santa Clara mitigation levy, for the active and passive coefficients (sloping backfill) Ka , Kp as well as the wall friction angle values. Since the flood walls are located on top of the levy, the hinge point is not far enough from the flood wall to assume a flat bedding surface per Caltrans memo 22-1. To assist you, the attached Caltrans memo 22-1 shows the geometry of the wall foundation we are using assuming 16" or 24" CIDH piles. The pile cap extends 3" on each side of the pile and is 2' deep. The max wall heights are expected to be 5' or so above the levy.

We are also investigating using L -Shaped spread footing alternative. For an 8.0' high flood wall including 2.5' cover and 1.5 thick footing and under hydrostatic pressure only, the footing width we are coming up with is 7' wide. What can we assume for uplift and seepage under this scenario?

Thank you for your assistance.

Majdi kanaan/Sac
916-920-0212 Ext 361

29169.tif

## IV. Foundation Design

The allowable vertical soil bearing capacity, the soil properties to be used in calculating the lateral soil bearing values and other pertinent foundation data will be shown in the foundation report that is provided by the Engineering Geology and Technical Services Branch of the Transportation Laboratory.

The passive soil pressures *shall be* increased by a factor of 1.5 for the design of laterally loaded piles located in cohesionless soils and in level ground. The increased pressure is defined as the 'EFFECTIVE' passive pressure and the increase factor of 1.5 is defined as the 'ISOLATION' factor. The 'ISOLATION' factor is a means to account for the assumption that a laterally loaded pile is resisted by a section of earth that is wider than that of the pile. A level ground condition is defined as one in which the ground surface is approximately level or, when sloping down and away from the pile, is not steeper than 10:1 for $\phi = 35°$ or 14:1 for $\phi = 25°$. A level ground condition may also be assumed when the hinge point of any adjacent down or negative slope that is steeper than 10:1 for $\phi = 35°$ or 14:1 for $\phi = 25°$ is not located closer than 2 times the pile embedment. The 10:1 and 14:1 negative slopes are approximately equal to a $\beta/\phi$ ratio of - 0.15, where $\beta$ is the angle of the slope in degrees and $\phi$ is the angle of shearing resistance in degrees. There shall be no increase in the 'EFFECTIVE' passive pressure for piles located in cohesive soils or in sloping ground. Figure 5 illustrates the parameter for the level ground condition.

With the exception of the factors of safety for the stability of spread footings, the foundation requirements for embedment, width, depth and strength shall be determined by the Load Factor Design (LFD) Method using the loading combinations, the load factors and the strength reduction factors shown in Section II: Load Combinations.

Piles located on slopes *are* to be protected by a berm. The berm should have 305 mm minimum width and provide 152 mm minimum depth of cover above the top of pile or pile cap.

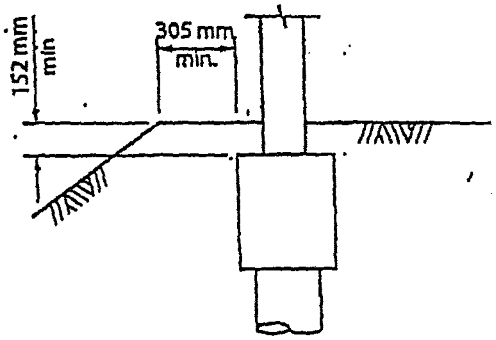


Figure 4

# APPENDIX D

October 30, 2002
<job02-029B\Baylands Soil Report1 .doc>

Mr. Stephen Huang
URS Corporation
55 South Market St., Suite 1650
San Francisco, CA 95113

SUBJECT: **Corrosion Engineering Services for the SCVWD Lower Guadalupe River Flood Protection Project – Baylands Weir**

Dear Mr. Huang:

V & A Consulting Engineers (V & A) was retained by URS Corporation (URS) to perform a soil investigation as part of the Lower Guadalupe Flood Protection Project. The objective of the investigation was to perform field-soil resistivity testing and chemical analysis of selected soil samples, and evaluate these results with respect to the possible levels of corrosion of buried concrete foundation materials, buried metallic structures, corrugated metal pipe, and prestressed concrete piles.

## TEST METHODS

In attempting to predict corrosion problems associated with a particular type of buried metallic or concrete structures prior to installation, it is necessary to investigate the soil conditions the structure will encounter. Since corrosion of metal is fundamentally an electrochemical process accompanied by current flow, the electrochemical characteristics of a soil are of primary importance when evaluating corrosivity. Test methods utilized during this investigation reflect the most practical methods of evaluating corrosivity.

### Soil Resistivity

Resistivity of the soil was measured by V & A at two locations at the project site on April 23, 2002. Figure 1 shows the vicinity and project location maps. Figure 2 shows an aerial view of the project site with the soil resistivity testing locations highlighted in blue.

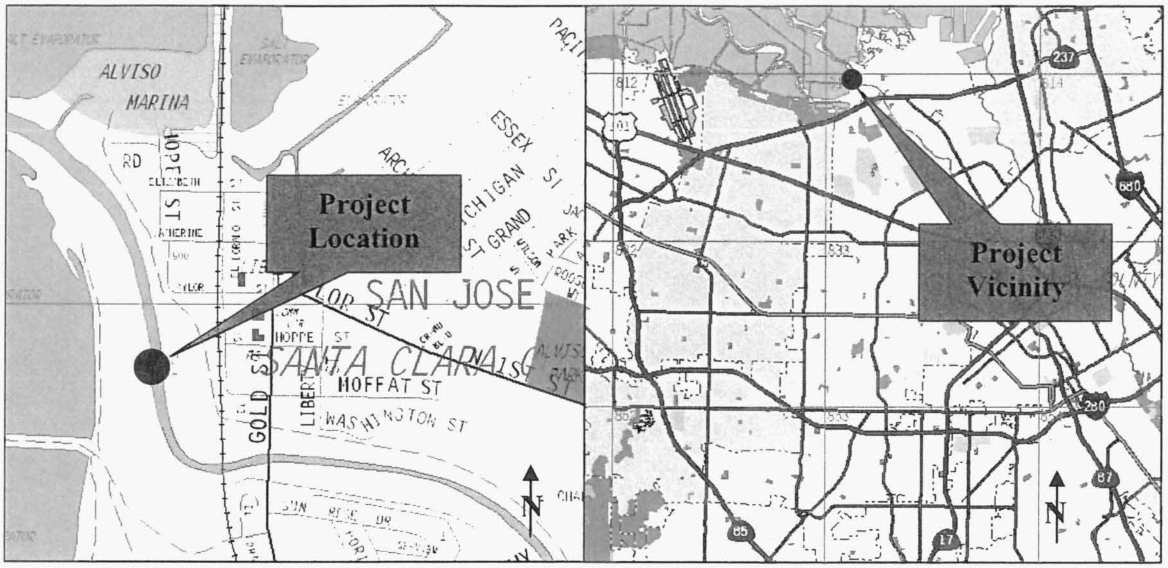
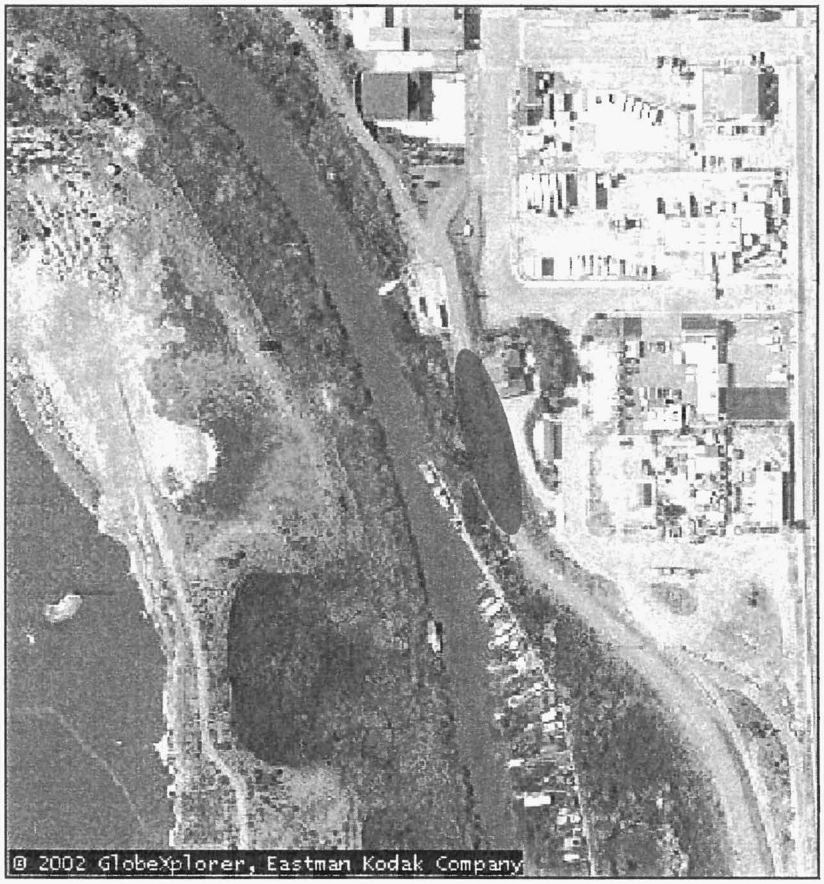


Figure 1. Vicinity and Project Site Maps for Baylands Section



© 2002 GlobeXplorer, Eastman Kodak Company

Figure2. Aerial Photograph of Soil Resistivity Testing Locations

Soil resistivity measurements were conducted by the Wenner 4-pin method, utilizing a Soil Resistance Meter, Model 400, manufactured by Nilsson Electrical Laboratory, Inc. The Wenner method involves the use of four metal probes or electrodes, driven into the ground along a straight line, equidistant from each other, as shown in Figure 3.

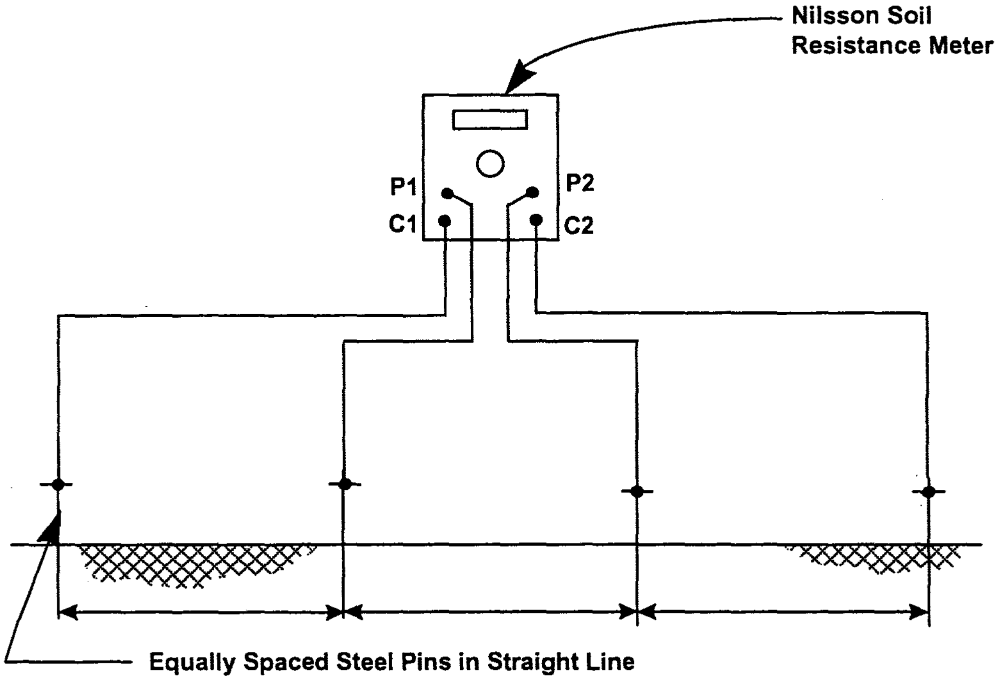


**Figure 3. Wenner 4-Pin Method for Measurement of Soil Resistivity**

An alternating current from the Soil Resistance Meter causes a current to flow through the soil between the outside probes, C1 and C2. Due to the resistance of the soil, the current creates a voltage gradient, which is proportional to the average resistance of the soil mass to a depth equal to the distance between probes. The voltage drop is then measured across pins, P1 and P2. Resistivity of the soil is then computed from the instrument reading according to the following formula:

$$p = 2\pi AR$$

where:

$p =$ soil resistivity (ohm - cm)
$A =$ distance between probes (cm)
$R =$ soil resistance, instrument reading (ohms)
$\pi = 3.14159$

Soil resistivity measurements were conducted at probe spacings of 5, 10, 15 and 20 feet. The resistivity values obtained and listed in Table 1 represent the average resistivity of the soil to a depth equal to the pin spacing and the resistivity values for the designated layer of soil.

## TEST RESULTS

Data obtained during the field resistivity testing has been summarized in tabular form for analysis and presentation. Table 1 lists the results of the field soil resistivity measurements conducted at the sites. Table 2 summarizes the results of the field soil resistivity measurements. Table 3 lists the "as-received" and minimum soil resistivity for each sample collected as well as the chemical analysis of the samples.

### Laboratory Analysis of Soil Samples

Laboratory analysis was performed on soil samples provided to V & A. The testing included laboratory resistivity in the "as-received" and minimum (or saturated) condition. The minimum or saturated resistivity provides information about the soil during wet weather conditions. With the addition of distilled water to each sample, the minimum resistivity will decrease from the original "as-received" soil resistivity, in almost every instance. This decrease in soil resistivity, when saturated, indicates a possible influence from additional chemical constituents. This change can be attributed to the following factors:

1. The presence of chemicals in the "as-received" soil going into solution and providing a less resistive path for current flow.

2. Large concentrations of bicarbonate ions which tend to lower the soil resistivity but are not considered corrosive to buried steel and ductile iron structures.

## Table 1. Field Soil Resistivity Test Data

| Site No. | Depth (meters) | R1 (ohm) | | "K" Factor | Resistivity (ohm-cm) | 1/R1 (mhos) | Del (1/R1) Del mhos | R2 (ohm) | "K" Factor (cm) | Layer Resistivity (ohm-cm) | Depth (meters) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Meter Reading | Mult | | | | | | | | |
| 1 | 1.5 | 1.9 | 1 | 958 | **1,820** | 0.5 | | | 0 | **1.820** | 0 - 1.5 |
| | 3 | 1.1 | 1 | 1.915 | **2,107** | 0.9 | 0.4 | 2.6 | 958 | **2.503** | 1.5 - 3 |
| | 4.6 | 6.0 | 0.1 | 2,873 | **1,724** | 1.7 | 0.8 | 1.3 | 958 | **1.265** | 3 - 4.6 |
| | 6.1 | 3.9 | 0.1 | 3,832 | **1,494** | 2.6 | 0.9 | 1.1 | 958 | **1,067** | 4.6 - 6.1 |
| 2 | 1.5 | 1.7 | 1 | 958 | **1,629** | 0.6 | | | 0 | **1.629** | 0 - 1.5 |
| | 3 | 1.3 | 1 | 1.915 | **2,490** | 0.8 | 0.2 | 5.5 | 958 | **5,293** | 1.5 - 3 |
| | 4.6 | 5.8 | 0.1 | 2.873 | **1,666** | 1.7 | 1.0 | 1.0 | 958 | **1,003** | 3 - 4.6 |
| | 6.1 | 3.6 | 0.1 | 3.832 | **1,380** | 2.8 | 1.1 | 0.9 | 958 | **909** | 4.6 - 6.1 |

## Table 2. Summary of Field Resistivity Results

| Depth | Minimum | Maximum | Average |
|---|---|---|---|
| At Depth Resistivity | | | |
| 1.5 m | 1,629 | 1,820 | 1,724 |
| 3.0 m | 2,107 | 2,490 | 2,298 |
| 4.6 m | 1,666 | 1,724 | 1,695 |
| 6.1 m | 1,380 | 1,494 | 1,437 |
| Layer Resistivity | | | |
| 0.0 - 1.5 m | 1,629 | 1,820 | 1,724 |
| 1.5 - 3 m | 2,503 | 5,293 | 3,898 |
| 3 - 4.6 m | 1,003 | 1,265 | 1,134 |
| 4.6- 6.1 m | 909 | 1,067 | 988 |

## Table 3. Soil Resistivity and Chemical Data

| Item No. | Boring No. | Sample No. | Depth (meters) | Resistivity (ohm-cm) | | pH | Chloride (ppm) | Sulfate (ppm) | Bicarbonate (ppm) |
|---|---|---|---|---|---|---|---|---|---|
| | | | | A.R. | Saturated | | | | |
| 1 | EB-28 | 5 - 2 | 6.4 | 720 | 540 | 7.2 | 790 | 335 | 918 |

Since corrosion is a natural electrochemical process accompanied by the flow of electrical current, it is important to understand how easily current will travel through a medium surrounding a metal object. Resistivity is a measure of the ability of a soil to conduct an electric current. The higher the resistivity the more difficult it is for the soil to conduct a current. Soil resistivity is primarily dependent on the chemical and moisture content of the soil. The higher the level of chemical constituents the lower the soil resistivity. Additional moisture generally decreases the soil resistivity up to the point where the maximum solubility for the dissolved ionic chemicals is achieved. Beyond this point, an increase in moisture generally increases the resistivity as the chemicals become more and more diluted. Since corrosion rate depends on current flow to and from a metal and the adjacent medium, the corrosion activity of steel reinforcement in soil normally increases as soil resistivity decreases. The following table correlates resistivity values with degree of corrosivity. The interpretation of corrosivity varies among corrosion engineers. However, this table is a generally accepted guide.

| **Soil Resistivity (ohm-cm)** | **Degree of Corrosivity** |
|---|---|
| 0 - 500 | Very Corrosive |
| 500 - 1,000 | Corrosive |
| 1,000 - 2,000 | Moderately Corrosive |
| 2,000 - 10,000 | Mildly Corrosive |
| Above 10,000 | Negligible |

*Reference: NACE Corrosion Basics, page 191*

Results of the soil laboratory analysis are shown in Table 3. A wide variety of soluble salts is typically found in soils. Two soils having the same resistivity may have significantly different corrosion characteristics, depending on the specific ions available. The major constituents which accelerate corrosion are chlorides, sulfates and the acidity (pH) of the soil. Calcium and magnesium tend to form insoluble oxide and bicarbonate precipitates, in basic environments, which can create a protective layer over the metal surface and reduce corrosion activity. Chloride ions tend to break down otherwise protective surface deposits, and can result in corrosion of buried metallic structures and reinforcing steel in concrete structures. Sulfates in soil can be highly aggressive to portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. High concentrations of bicarbonates tend to decrease soil resistivities. However, bicarbonates are not aggressive to buried steel and concrete. Although bicarbonates are not aggressive, lower resistivity environments can promote corrosion activity.

The following tables correlate the effect of chlorides on the rate of corrosion of steel, ductile iron or reinforcing steel in concrete, and the effect of sulfates on concrete:

| Water Soluble Chloride Concentration (ppm) | Degree Of Corrosivity |
|---|---|
| Over 5,000 | Severe |
| 1,500 - 5,000 | Considerable |
| 500 - 1,500 | Corrosive |
| Below 500 | Threshold |

*Reference: Extrapolation from California Test Method 532, Method for Estimating The Time To Corrosion of Reinforced Concrete Substructures and V & A Consulting Engineers' experience*

The water soluble chloride level was 790 ppm. This level is considered to be in the corrosive category. The water soluble sulfate level was 335 ppm. This level is considered to be moderately corrosive.

| Water Soluble Sulfate Concentration (ppm) | Degree Of Corrosivity |
|---|---|
| Over 10,000 | Very Severe |
| 1,500 - 10,000 | Severe |
| 150 - 1,500 | Moderate |
| 0 - 150 | Negligible |

*Reference: ACI-318, Building Code Requirements for Reinforced Concrete*

Acidity, as indicated by the pH value, is another important factor of soil. The lower the pH (the more acidic the environment), the higher will be the corrosivity with respect to buried metallic and concrete structures. As pH increases above 7 (the neutral value), conditions become increasingly more alkaline and passive to buried steel structures. V & A has developed a table correlating the effect of pH on the rate of corrosion of buried steel or concrete structures. The data is derived from V & A's extensive experience and review of the literature, e.g., Romanoff M., Underground Corrosion, and Uhlig H., Corrosion and Corrosion Control.

| pH | Degree Of Corrosivity |
|---|---|
| < 5.5 | Severe |
| 5.5-6.5 | Moderate |
| 6.5-7.5 | Neutral |
| > 7.5 | Negligible |

The pH of the tested boring sample was 7.2. This level is considered to be a neutral value. The level of bicarbonate ions was 918 ppm. High concentrations of bicarbonates tend to decrease soil resistivities. Bicarbonates are not aggressive to buried steel or concrete, but the lower resistivity environments can promote corrosion activity.

The saturated (minimum) soil resistivity was 540 ohm-cm. The soil is considered to be corrosive at this location.

## RECOMMENDATIONS

Based upon the soil corrosivity data and review of the project requirements, corrosion monitoring and control will be required to prevent corrosion of metallic and reinforced structures. These recommendations are in accordance with Caltrans Bridge Design Specifications, Section 8.22. The quality of materials and workmanship should be inspected during construction by a corrosion engineer

### Prestressed Concrete Piles

Prestressed concrete piles should be constructed of durable concrete such as described in ACI Standards 201.2R and 222R. These recommendations include but are not limited to the following:

- The water/cement ratio should not exceed 0.40.
- A concrete cover of a minimum of 2 inches should be applied over all steel reinforcement.
- Mineral admixtures conforming to ASTM Designation C1240 and/or ASTM Designation C618 Type F and/or N should be used.
- Salt-free sand and potable water should be used.
- The concrete should be allowed to cure according to the manufacturer's recommendations.

### Buried Metallic Structures

Based on the low soil resistivity it is recommended that buried metallic piping systems, such as steel and ductile iron, be coated with a bonded dielectric or mortar coating. All non-welded joints should be bonded to provide electrical continuity. Cathodic protection should be installed to provide corrosion protection. Buried metallic piping should be electrically isolated from other structures, or where a change in piping or coating material occurs.

### Buried Reinforced Concrete Structures

Buried reinforced concrete structures should be constructed of durable concrete such as described in ACI Standards 201.2R and 222R. These recommendations include but are not limited to the following:

- The water/cement ratio should not exceed 0.40.
- A concrete cover of a minimum of 3 inches should be applied over all steel reinforcement.
- Salt-free sand and potable water should be used.
- The concrete should be allowed to cure according to the manufacturer's recommendations.

## MSE Retaining Walls

For permanent structures, the reinforcement elements shall be designed to have a minimum design life of 75 years. For the metallic reinforcement, galvanization loss will be:

- 15 µm/year/side for the first 2 years
- 4 µm/year/side for subsequent years

The loss of bare steel after the galvanized zinc is depleted will be 12 µm/year/side. These estimates are from the AASHTO Standard Specifications for Highway Bridges, Section 5.8.6.1. It is assumed that the soil backfill used is nonaggressive (i.e. Class 1 Backfill), meeting the following criteria:

- pH of 5 to 10
- Resistivity of not less than 3,000 ohm-cm
- Chlorides not greater than 100 ppm
- Sulfates not greater than 200 ppm.

The above mentioned thicknesses are not valid if the following conditions are present:

- The MSE wall is exposed to marine or other chloride rich environments
- The MSE wall will be exposed to stray currents such as from nearby underground power lines or adjacent railways
- The backfill material is aggressive

Please call if you have any questions or comments.

Sincerely,
**V & A CONSULTING ENGINEERS**
*The Infrastructure Preservation People*

for.

Glenn H. Willson, PE
Project Manager